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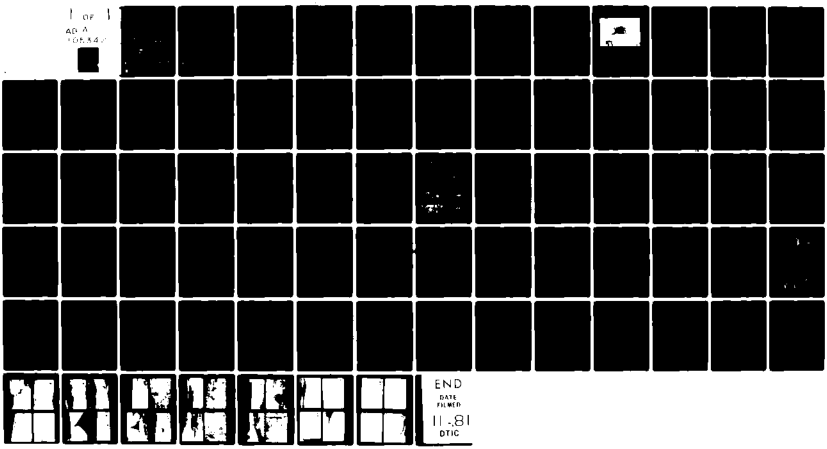

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VON DER AHE DAM
FRANKLIN COUNTY, MISSOURI
MO 31077

PHASE 1 INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

United States Army
Corps of Engineers
...Serving the Army
...Serving the Nation

St. Louis District

PREPARED BY: U.S. ARMY ENGINEER DISTRICT, ST. LOUIS

FOR: STATE OF MISSOURI

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Dam Safety, Lake, Dam Inspection, Private Dams

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report was prepared under the National Program of Inspection of Non-Federal Dams. This report assesses the general condition of the dam with respect to safety, based on available data and on visual inspection, to determine if the dam poses hazards to human life or property.

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DEPARTMENT OF THE ARMY
ST. LOUIS DISTRICT, CORPS OF ENGINEERS
210 NORTH 12TH STREET
ST. LOUIS, MISSOURI 63101

IN REPLY REFER TO

SUBJECT: Von Der Ahe Dam Phase I Inspection Report

This report presents the results of field inspection and evaluation of the Von Der Ahe Dam.

It was prepared under the National Program of Inspection of Non-Federal Dams.

This dam has been classified as unsafe, non-emergency by the St. Louis District as a result of the application of the following criteria:

1) Spillway will not pass 50 percent of the Probable Maximum Flood without overtopping the dam.
2) Overtopping of the dam could result in failure of the dam.
3) Dam failure significantly increases the hazard to loss of life downstream.

SUBMITTED BY: SIGNED — 31 JUL 1980
Chief, Engineering Division — Date

APPROVED BY: SIGNED — 31 JUL 1980
Colonel, CE, District Engineer — Date

VON DER AHE DAM

FRANKLIN COUNTY, MISSOURI

MISSOURI INVENTORY NO. 31077

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

Prepared by

Anderson Engineering, Inc. Springfield, Missouri
Hanson Engineers, Inc., Springfield, Illinois

Under Direction of

St. Louis District, Corps of Engineers

For

Governor of Missouri

June, 1980

PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam:	Von Der Ahe Dam
State Located:	Missouri
County Located:	Franklin County
Stream:	Tributary of Calvey Creek
Date of Inspections:	25 June 1979 22 May 1980

Von Der Ahe Dam was inspected by an interdisciplinary team of engineers from Anderson Engineering, Inc. of Springfield, Missouri and Hanson Engineers, Inc. of Springfield, Illinois. The purpose of the inspection was to make an assessment of the general condition of the dam with respect to safety, based upon available data and visual inspection, in order to determine if the dam poses hazards to human life or property.

The guidelines used in the assessment were furnished by the Department of the Army, Office of the Chief of Engineers, and they have been developed with the help of several Federal and State agencies, professional engineering organizations, and private engineers. Based on these guidelines, this dam has been classified by the St. Louis District Corps of Engineers as a small size dam with a high downstream hazard potential. The estimated damage zone extends approximately 2 miles downstream of the dam. Located within this zone are 6 dwellings and 8 mobile homes.

Our inspection and evaluation indicates that the combined spillways do not meet the criteria set forth in the guidelines for a dam having the above size and hazard potential. The combined spillways will pass 33 percent of the Probable Maximum Flood without overtopping. The Probable Maximum Flood is defined as the flood discharge that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. The guidelines require that a dam of small size with a high downstream hazard potential pass 50 to 100 percent of the PMF. Considering the volume of water impounded, the height of the dam and the large flood plain downstream, 50 percent of the PMF has been determined to be the appropriate spillway design flood. The 100-year frequency flood will not overtop the dam. The 100-year flood is one that has a 1 percent chance of being exceeded in any given year.

The embankment appeared to be generally in good condition. Deficiencies visually observed by the inspection team were: (1) Erosion of downstream face at toe of embankment; (2) Erosion of upstream face of dam at elevation of primary spillway inlet; (3) Lack of trash rack on the primary spillway inlet; and (4) Apparent seepage through bedrock in the east abutment; (5) Seepage through drain pipe valve. Another deficiency was the lack of seepage and stability analysis records.

It is recommended that the owners take the necessary action in the near future to correct the deficiencies reported herein. A detailed discussion of these deficiencies is included in the following report.

Dave Daniels

Dave Daniels, P.E.
Hanson Engineers, Inc.

Nelson Morales, P.E.
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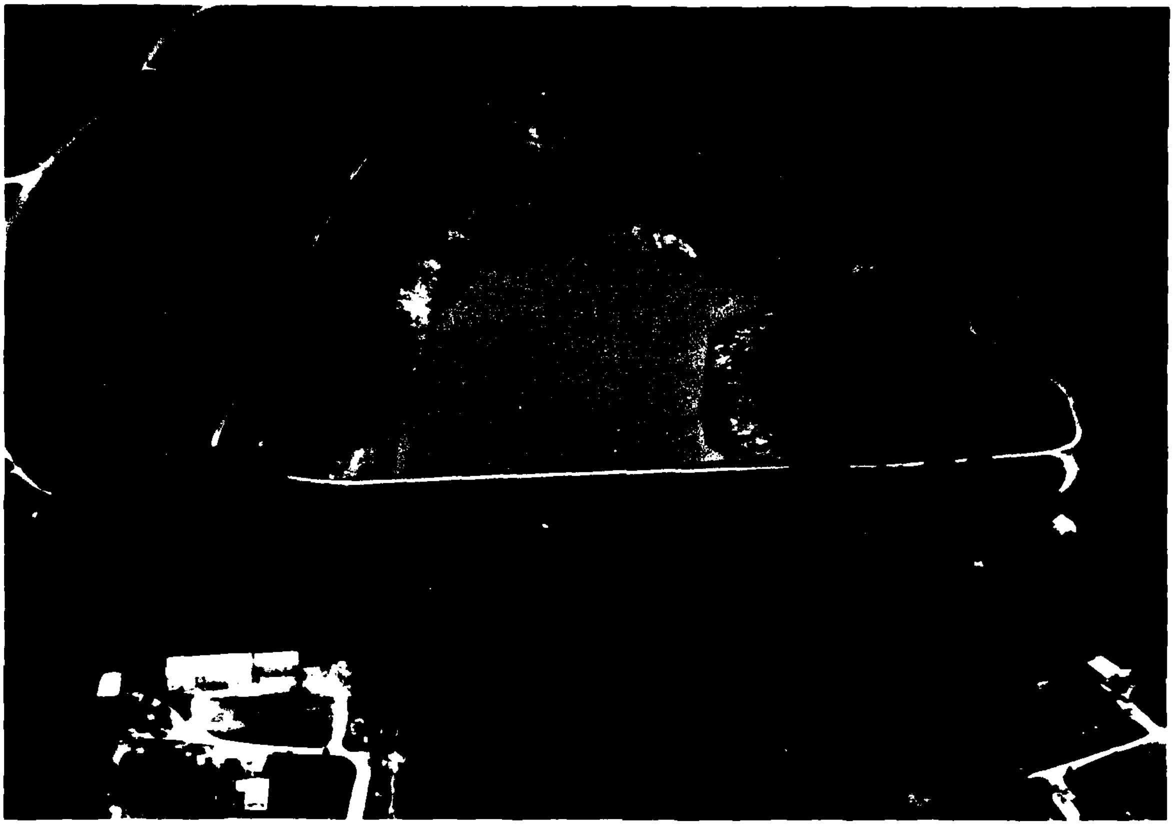

AERIAL VIEW OF LAKE AND DAM

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

VON DER AHE DAM ID No. 31077

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SECTION 1 - PROJECT INFORMATION

1.1 GENERAL:

A. Authority:

The National Dam Inspection Act, Public Law 92-367, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a program of safety inspection of dams throughout the United States. Pursuant to the above, the St. Louis District, Corps of Engineers, District Engineer directed that a safety inspection be made of Von Der Ahe Dam in Franklin County, Missouri.

B. Purpose of Inspection:

The purpose of the inspection was to make an assessment of the general condition of the dam with respect to safety, based upon available data and a visual inspection in order to determine if the dam poses hazards to human life or property.

C. Evaluation Criteria:

Criteria used to evaluate the dam were furnished by the Department of the Army, Office of the Chief Engineers, "Recommended Guidelines for Safety Inspection of Dams, Appendix D." These guidelines were developed with the help of several federal agencies and many state agencies, professional engineering organizations, and private engineers.

1.2 DESCRIPTION OF PROJECT:

A. Description of Dam and Appurtenances:

The Von Der Ahe Dam is an earth fill structure approximately 37 ft. high and 660 ft. long at the crest. The appurtenant works consist of an 18 inch diameter concrete pipe at east abutment, a 24 inch diameter steel primary spillway pipe at Sta. 4 + 82 and a 9 inch diameter steel drawdown pipe at Sta. 4 + 95. Sheet 3 of Appendix A shows a plan, profile and typical section of the embankment.

B. Location:

The dam is located in the east central part of Franklin County, Missouri on a tributary of Calvey Creek. The dam and lake are within the Lonedell, Missouri 7.5 minute quad-

rangle sheet (Section 24, T42N, R2E - latitude 38 21.7'; longitude 90 45.1'). Sheet 2 of Appendix A shows the general vicinity.

C. Size Classification:

With an embankment height of 37 ft. and a maximum storage capacity of approximately 328 acre-ft., the dam is in the small size category.

D. Hazard Classification:

The St. Louis District, Corps of Engineers has classified this dam as a high hazard dam. The estimated damage zone extends approximately 2 miles downstream of the dam. Located within this zone are 6 dwellings and 8 mobile homes.

E. Ownership:

The dam is owned by Mr. Russell L. Von Der Ahe. The owner's address is 600 Rudder Ave., Fenton, Missouri 63026.

F. Purpose of the Dam:

The dam was constructed primarily for recreational purposes.

G. Design and Construction History:

The design information obtained for this dam is as shown on Sheets 4 through 6 of Appendix A. Construction of the dam was completed by William Tyree Excavating, Sullivan, Missouri in the summer of 1977. The construction is patterned after similar dams designed by the Soil Conservation Service. The material for the embankment was obtained from the lake area. The owner stated that the dam commenced to leak in the fall of 1978. Leakage from the dam surfaced several hundred yards downstream. A trench was excavated near the downstream toe to determine if the dam was leaking through the embankment. A negligible amount of water entered the trench through the embankment. At this time the drawdown pipe was used to assist in lowering the pool level. Technical assistance was obtained from Mr. Thomas Dean of the Missouri Department of Natural Resources. Reports written by Mr. Dean are included in Appendix A. At the time of inspection, excavation to bedrock in the lake area near the east abutment was completed. This exposed rock revealed separation between two different formations. This appeared to be a fairly massive sandstone over a thinly bedded and cracked limestone. The rock had been blasted back into the east abutment. The separation at this point appeared small.

Subsequent to our site inspection a grout curtain was drilled by Test Drilling Service Co. of St. Louis, Missouri. The line of grout holes were drilled through the embankment from the east abutment westward to the excavated pit at the toe of the upstream face (Sta. 2 + 35). Mr. Tom Dean of the Missouri Department of Natural Resources said the grout holes were drilled generally on the front face of the dam. On July 27, 1979, Mr. Tyree said the grouting program was completed and the excavated area would be filled in and the lake allowed to fill.

H. Normal Operative Procedures:

The uncontrolled flow from the lake is passed through the 24 inch diameter spillway pipe near the center of the embankment, an earth cut spillway in the west abutment and an 18 inch diameter concrete pipe in the east abutment. The pool level has been up to the 24 inch diameter pipe and subsequently drained due to apparent leakage.

1.3 PERTINENT DATA:

Pertinent data about the dam, appurtenant works, and reservoir are presented in the following paragraphs. Sheet 3 of Appendix A presents a plan, profile and typical section of the embankment.

A. Drainage Area:

The drainage area for this dam, as obtained from the U.S.G.S. quad sheet, is equal to approximately 292 acres.

B. Discharge at Dam Site:

(1) All discharge at the dam site is through uncontrolled spillways.

(2) Estimated Total Spillway Capacity at Maximum Pool (Top of Dam - El. 104.3): 701 cfs

(3) Estimated Capacity of Primary Spillway: 22 cfs

(4) Estimated Experienced Maximum Flood at Dam Site: Unknown

(5) Diversion Tunnel Low Pool Outlet at Pool Elevation: Not Applicable

(6) Diversion Tunnel Outlet at Pool Elevation: Not Applicable

(7) Gated Spillway Capacity at Pool Elevation: Not Applicable

(8) Gated Spillway Capacity at Maximum Pool Elevation: Not Applicable

C. Elevations: (See Sheet 3, Appendix A for benchmark)

(1) Top of Dam: 104.3 Feet (Low Point); 105.5 Feet (Ave.)

(2) Principal Spillway Crest: 98.0 Feet

(3) Emergency Spillway Crest: 102.2 Feet

(4) Principal Outlet Pipe Invert: 98.0 Feet

(5) Streambed at Centerline of Dam: 70.0 Feet

(6) Pool on Date of Inspection: 70.0 Feet (Empty)

(7) Apparent High Water Mark: 97.6 Feet

(8) Maximum Tailwater: Unknown

(9) Upstream Portal Invert Diversion Tunnel: Not Applicable

(10) Downstream Portal Invert Diversion Tunnel: Not Applicable

D. Reservoir Lengths:

(1) At Top of Dam: 2180 Feet

(2) At Principal Spillway Crest: 1800 Feet

(3) At Emergency Spillway Crest: 1990 Feet

E. Storage Capacities:

(1) At Principal Spillway Crest: 187 Acre-Feet

(2) At Top of Dam: 328 Acre-Feet

(3) At Emergency Spillway Crest: 282 Acre-Feet

F. Reservoir Surface Areas:

(1) At Principal Spillway Crest: 20 Acres

(2) At Top of Dam: 25 Acres (Low Point), 27 Acres (High Point)

(3) At Emergency Spillway Crest: 23 Acres

G. Dam:

(1) Type: Earth Fill

(2) Length at Crest: 660 Feet

(3) Height: 37 Feet

(4) Top Width: 15 Feet

(5) Side Slopes: Upstream 3.0H to 1.0V; Downstream 3.0H to 1.0V (See Sheet 3 of Appendix A)

(6) Zoning: Homogeneous - No Internal Drainage (Information obtained from Mr. Persley, the caretaker)

(7) Impervious Core: None (Information obtained from Mr. Persley, the caretaker)

(8) Cutoff: Key Trench to Rock (Information obtained from Mr. Persley, the caretaker)

(9) Grout Curtain: See Paragraph 1.2G.

H. Diversion and Regulating Tunnel:

(1) Type: None

(2) Length: Not Applicable

(3) Closure: Not Applicable

(4) Access: Not Applicable

(5) Regulating Facilities: Not Applicable

I. Spillway

I.1 Principal Spillway:

(1) Location: Approximately center of dam

(2) Type: 24 inch diameter steel pipe (uncontrolled)

I.2 Emergency Spillways:

(1) Location: West abutment and East abutment.

(2) Type: Cut into Natural Earth in the West abutment and an 18 inch diameter concrete pipe at the East abutment.

J. Regulating Outlets:

A 9 inch diameter steel pipe is located in the embankment at Sta. 4 + 95 for drawdown purposes. The valve for the pipe is located on the downstream face of the dam adjacent to the outlet (See photo No. 10, 15 & 16 - Appendix D).

SECTION 2 - ENGINEERING DATA

2.1 DESIGN:

The Soil Conservation Service provided some preliminary design assistance for Von Der Ahe Dam. Sheet 4, 5 & 6 of Appendix A contain some design information on the primary and emergency spillway. No other design computations or reports could be located. No documentation of construction inspection records have been obtained. To our knowledge there are no documented maintenance data.

A. Surveys:

No information regarding pre-construction surveys are able to be obtained. Sheet 3 of Appendix A presents a plan, profile and cross section of the dam from survey data obtained during the site inspection. The top of the inlet of the 24 inch diameter steel pipe primary spillway was used as a site datum of assumed elevation 100.00 (See Sheet 3, Appendix A).

B. Geology and Subsurface Materials:

The site is located near the northeastern limit of the Ozarks geologic region of Missouri. The Ozarks are characterized topographically by hills, plateaus and deep valleys. The most common bedrock types are dolomite, sandstone and chert. The Missouri Geological Survey indicates that the bedrock in the site area consists primarily of the Jefferson City formation of the Canadian Series in the Ordovician System. The Jefferson City formation is composed principally of light brown to brown medium to fine crystalline dolomite and argillaceous dolomite. The publication "Caves of Missouri" indicates that while numerous caves are known to exist in Franklin County, they are densely clustered in the south-central part of the county, at least 15 miles from the site.

The "Geologic Map of Missouri" indicates a normal fault passing about 6 miles northwest of the site in a north-south direction. The Missouri Geological Survey has indicated that the faults in this area are generally considered to be inactive and have been for several hundred million years.

The soils overlying the Jefferson City formation are of the Union-Fullerton-McGirk Soil Association, and consist of a veneer of clayey residual material with a thin cover of loess. The loessial deposits in upland areas are generally

about 5 feet thick. The soils used in the embankment appear to all fall within the Unified Soil Classification of CL.

C. Foundation and Embankment Design:

No design computations are available. Information from Mr. Persley, the owner's caretaker, indicates that the dam is composed of materials taken from the lake area upstream of the dam. A core trench to bedrock was incorporated under the dam. The maximum depth of the trench was approximately 20 feet. The trench was filled using clay taken from the upstream lake area. No internal drainage features were incorporated, nor is there any particular zoning of the embankment. No construction inspection records are available. The information contained in this paragraph was obtained from Mr. Persley.

D. Hydrology and Hydraulics:

Included as Sheets 5 & 6 of Appendix A are the design calculations for the emergency and primary spillways as prepared by the Soil Conservation Service. Our analyses of the PMF are presented in Appendix C. These analyses were based on our field survey and observations, and estimates of areas and volumes from the U.S.G.S. quad sheet. It was concluded that the structure will pass 33 per cent of the Probable Maximum Flood without overtopping. The 100-year frequency flood will not overtop the dam.

E. Structure:

The appurtenant structures for the dam are the 9 inch diameter drawdown pipe with the valve at the downstream end, the 24 inch diameter primary spillway pipe and the 18 inch diameter concrete culvert pipe. The drawdown pipe has 3 anti-seep collars as indicated by Mr. Persley. The drawdown pipe was used to lower the lake when the leakage from the lake was observed. Design information available is included in Appendix A.

2.2 CONSTRUCTION:

No construction inspection data have been obtained.

2.3 OPERATION AND MAINTENANCE:

There are no operating records to our knowledge. The dam has not been in existence long enough for vegetation to be a problem.

2.4 EVALUATION:

A. Availability:

The engineering data available are as listed in Section 2.1.

B. Adequacy:

The engineering data available were inadequate to make a detailed assessment of the design, construction, and operation. Seepage and stability analyses comparable to the requirements of the "Recommended Guidelines for Safety Inspection of Dams" were not available, which is considered a deficiency. These seepage and stability analyses should be performed for appropriate loading conditions (including earthquake loads) and made a matter of record.

C. Validity:

No valid engineering data on the design or construction of the embankment are available to our knowledge.

SECTION 3 - VISUAL INSPECTION

3.1 FINDINGS:

A. General:

The field inspection was made on 25 June 1979. The inspection team consisted of personnel from Anderson Engineering, Inc. of Springfield, Missouri and Hanson Engineers, Inc. of Springfield, Illinois. The team members were:

David Daniels-Hanson Engineers, Inc. (Geotechnical Engineer)
Nelson Morales - Hanson Engineers, Inc. (Hydraulic Engineer)
Steven L. Brady - Anderson Engineering, Inc. (Civil Engineer)
Tom Beckley - Anderson Engineering, Inc. (Civil Engineer)

B. Dam:

At the time of inspection there was no water present in the lake. The embankment appears to be generally in good condition. The centerline of the dam was constructed straight with an obvious crown across the crest. Some surface cracking was observed along the top of the dam although this did not appear to be serious. The cracks were thin shrinkage cracks of the soil crust. The cracks were approximately 1/16 inch wide and 1/4 inch deep. No sloughing of the embankment was noted. Minor erosion along the length of the downstream toe of the embankment was observed. This erosion appeared to be recent. Some erosion due to wave action was noted on the upstream face at the level of the primary spillway. No riprap was used on the upstream face.

At Sta. 2 + 35 at the toe of the upstream face an excavation was open down into bedrock. This excavation is part of the remedial work to stop seepage through the east abutment as described in paragraph 1.2G. An area of reeds and cattails were observed several hundred yards downstream from the dam. As per the investigation made by the Missouri Department of Natural Resources this area is where the leakage from the lake is surfacing.

No instrumentation (monuments, piezometers, etc.) was observed.

C. Appurtenant Structures:

C.1 Primary Spillway:

The 24 inch diameter spillway is clear at both inlet and outlet. Some erosion is present around the inlet of the

pipe. The outlet at the downstream toe is approximately 2 feet above natural bedrock. The inlet does not have a trash rack.

C.2 Emergency Spillways:

The emergency spillways are in the west and east abutments. The spillways downstream are relatively free of brush and vegetation and are protected from the toe of the dam. To our knowledge the spillways have never been used.

D. Reservoir:

The slopes adjacent to the lake are moderate and no sloughing or serious erosion was noted. The watershed has some woods but is primarily pastureland.

E. Downstream Channel:

The slope of the channel is slight to moderate. The streambed is relatively clear of trees and debris. At approximately 150 feet downstream of the primary spillway outlet the channel is in bedrock.

3.2 EVALUATION:

The erosion areas on the downstream face of the embankment at the toe should be corrected and maintained. Erosion caused by wave action of the upstream face should be corrected and maintained.

After the repairs are completed and the lake is allowed to reach normal pool level, the leakage areas should be investigated by an engineer experienced in the design and construction of dams.

Because the valve of the lake drain is located on the downstream face of the dam, the full head of water impounded by the dam is acting entirely through the dam. The area around the lake drain outlet should be periodically inspected for seepage which might indicate a leak or rupture of the drain pipe which could eventually initiate a piping failure through the embankment. A trash rack should be installed on the primary spillway inlet.

Photographs of the dam, appurtenant structures, and the reservoir are presented in Appendix D.

SECTION 3A - VISUAL INSPECTION

3A.1 FINDINGS:

A. General:

An additional field inspection was made on 22 May 1980. The inspection team consisted of personnel from Anderson Engineering, Inc. of Springfield, Missouri and Hanson Engineers, Inc. of Springfield, Illinois. The team members were:

David Daniels-Hanson Engineers, Inc. (Geotechnical Engineer)
Gene Wertepny - Hanson Engineers, Inc. (Hydraulic Engineer)
Steven L. Brady - Anderson Engineering, Inc. (Civil Engineer)
Tom Beckley - Anderson Engineering, Inc. (Civil Engineer)

B. Dam:

At the time of inspection the lake was partially filled with water. The embankment appears to be generally in good condition. The centerline of the dam was constructed straight with an obvious crown across the crest. Some surface cracking was observed along the top of the dam although this did not appear to be serious. No sloughing of the embankment was noted. Minor erosion along the length of the downstream toe of the embankment was observed. This erosion does not appear any worse than when observed on 25 June 1979. Some erosion due to wave action was noted on the upstream face at the level of the primary spillway. No riprap was used on the upstream face.

The excavation at the east end of the upstream face as discussed in Section 3 was completed. This excavation was part of an attempt to stop seepage through the east abutment as described in paragraph 1.2G. No seepage was noted in the area downstream of the dam where previous investigations by the Missouri Department of Natural Resources had indicated the leakage from the lake had been surfacing.

Water was coming from the dewatering pipe as shown in Photo No. 27.

No instrumentation (monuments, piezometers, etc.) was observed.

C. Appurtenant Structures:

C.1 Primary Spillway:

The 24 inch diameter spillway is clear at both inlet and outlet. Some erosion is present around the inlet of the pipe. The outlet at the downstream toe is approximately 2 feet above natural bedrock. The inlet does not have a trash rack.

C.2 Emergency Spillways:

The emergency spillways are in the west and east abutments. The spillways downstream are relatively free of brush and vegetation and are protected from the toe of the dam. To our knowledge the spillways have never been used.

D. Reservoir:

The slopes adjacent to the lake are moderate and no sloughing or serious erosion was noted. The watershed has some woods but is primarily pastureland.

E. Downstream Channel:

The slope of the channel is slight to moderate. The streambed is relatively clear of trees and debris. At approximately 150 feet downstream of the primary spillway outlet the channel is in bedrock.

3A.2 EVALUATION:

The erosion areas on the downstream face of the embankment at the toe should be corrected and maintained. Erosion caused by wave action of the upstream face should be corrected and maintained.

After the lake is allowed to reach normal pool level, the previously reported leakage areas should be investigated by an engineer experienced in the design and construction of dams.

Because the valve of the lake drain is located on the downstream face of the dam, the full head of water impounded by the dam is acting entirely through the dam. The area around the lake drain outlet should be periodically inspected for seepage which might indicate a leak or rupture of the drain pipe which could eventually initiate a piping failure through the embankment. The valve on the dewatering pipe should be checked to see if it is leaking or not fully

closed. A trash rack should be installed on the primary spillway inlet.

Photographs of the dam, appurtenant structures, and the reservoir taken during the second inspection are presented in Appendix D.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 PROCEDURES:

The controlled outlet works for this dam is the 9 inch diameter drawdown pipe. The primary and emergency spillways are uncontrolled, so that the pool, when filled, will normally be controlled by rainfall, runoff and evaporation.

4.2 MAINTENANCE OF DAM:

With the corrective actions on the seepage problem now in progress, no maintenance of the dam has been performed.

4.3 MAINTENANCE OF OPERATING FACILITIES:

The drawdown facilities appear to be in good condition and at the present time regular maintenance has not been scheduled.

4.4 DESCRIPTION OF ANY WARNING SYSTEM IN EFFECT:

The inspection team is unaware of any existing warning system for this dam.

4.5 EVALUATION:

The area around the valve of the lake drain pipe should be periodically inspected for seepage which might indicate a leak or rupture of the drain pipe and initiate a piping failure through the embankment.

Trees and brush should be cut annually and erosional areas should be maintained. The emergency spillway and primary spillway should be periodically cleared of wood, debris and vegetation. The dam should be periodically inspected to detect possible seepage under or through the embankment.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES:

A. & B. Design and Experience Data:

The hydraulic and hydrologic analyses were based on: (1) a field check of spillway dimensions and embankment elevations; and (2) an estimate of the pool and drainage areas from the U.S.G.S. quad sheet. Hydraulic design calculations for the primary and emergency spillways were obtained from the Soil Conservation Serv. and are included in Appendix A.

C. Visual Observations:

The approach to the spillways are free of any brush or undergrowth. A trash rack for the primary spillway should be provided. The spillway downstream is relatively free of debris and vegetation and should be inspected periodically. The spillway is well away from the dam, and emergency spillway release would not be expected to endanger the dam. The emergency spillway has not been used according to the owner.

D. Overtopping Potential:

Based on the hydrologic and hydraulic analysis presented in Appendix C, the combined spillways will pass 33 percent of the Probable Maximum Flood. The Probable Maximum Flood is defined as the flood discharge that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. The recommended guidelines from the Department of the Army, Office of the Chief Engineers, require that this structure (small size with high downstream potential) pass 50 percent to 100 percent of the PMF, without overtopping. Considering the volume of water impounded, the height of the dam and the large flood plain downstream, 50 percent of the PMF has been determined to be the appropriate spillway design flood. The structure will pass a 100-year frequency flood without overtopping.

The routing of 50 percent of the PMF through the spillways and dam indicates that the dam will be overtopped by 0.98 ft. at elevation 105.28. The duration of the overtopping will be 1.17 hours and the maximum outflow will be 1817 cfs. The maximum discharge capacity of the combined spillways is 701 cfs.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY:

A. Visual Observations:

Visual observations which could adversely affect the structural stability of this dam are discussed in Sections 3.1B and 3.2.

B. Design and Construction Data:

No design and construction data for the foundation and embankment were available. On site inspection indicates that the materials composing the dam are primarily clayey residual soils. Mr. William Tyree, the contractor, has indicated that a clay key was incorporated under the dam. It is not known whether internal drainage features were incorporated. No construction inspection records are available. Seepage and stability analyses comparable to the requirements of the guidelines were not available, which constitutes a deficiency which should be rectified.

C. Operating Records:

No operating records have been obtained.

D. Post-Construction Changes:

The inspection team is not aware of any post-construction changes to the dam, other than the current remedial measures underway to correct the leakage along the east abutment.

E. Seismic Stability:

The structure is located in seismic zone 1. An earthquake of this magnitude would not generally be expected to cause severe structural damage to a well constructed earth dam of this size. However, it is recommended that the prescribed seismic loading for this zone be applied in stability analyses for this dam.

SECTION 7 - ASSESSMENT/REMEDIAL MEASURES

7.1 DAM ASSESSMENT:

This Phase I inspection and evaluation should not be considered as being comprehensive since the scope of work contracted for is far less detailed than would be required for an in-depth evaluation of dams. Latent deficiencies, which might be detected by a totally comprehensive investigation, could exist.

A. Safety:

The embankment is generally in good condition. Several items were noted during the visual inspection which should be investigated further, corrected or controlled. These items are: (1) minor erosion along downstream face of dam near the toe; (2) erosion along upstream face of dam at the level of the primary spillway pipe; (3) lack of any trash preventative measures at the inlet of the primary spillway pipe; (4) apparent leakage through bedrock in east abutment; and (5) seepage through drain pipe valve.

The dam will be overtopped by flows in excess of 33 percent of the Probable Maximum Flood. Overtopping of an earthen embankment could cause serious erosion and could possibly lead to failure of the structure.

B. Adequacy of Information:

The conclusions in this report were based on review of the information listed in Section 2.1, the performance history as related by others, and visual observation of external conditions. The inspection team considers that these data are sufficient to support the conclusions herein. Seepage and stability analyses comparable to the "Recommended Guidelines for Safety Inspection of Dams" were not available, which is considered a deficiency.

C. Urgency:

The remedial measures recommended in paragraph 7.2 should be accomplished in the near future. If the deficiencies listed in paragraph 7.1A are not corrected, and if good maintenance is not provided, the embankment condition will continue to deteriorate and possibly could become serious in the future. Priority should be given to increasing the size of the spillway and/or height of the dam.

D. Necessity for Phase II:

Based on the result of the Phase I inspection, no Phase II inspection is recommended.

E. Seismic Stability:

The structure is located in seismic zone 1. An earthquake of this magnitude would not generally be expected to cause severe structural damage to a well constructed earth dam of this size. However, it is recommended that the prescribed seismic loading for this zone be applied in any stability analyses performed for this dam.

7.2 REMEDIAL MEASURES:

The following remedial measures and maintenance procedures are recommended. All remedial measures should be performed under the guidance of a professional engineer experienced in the design and construction of dams.

(1) Spillway size and/or height of dam should be increased to pass 50 percent of the PMF. In either case, the spillway should be protected to prevent erosion.

(2) Seepage and stability analyses comparable to the requirements of the recommended guidelines should be performed by an engineer experienced in the design and construction of dams.

(3) Erosional areas along the downstream toe should be corrected and maintained.

(4) Erosional areas on the upstream face at normal pool level should be corrected and maintained. Consideration should be given to the installation of an erosional control system such as riprap along the face prior to refilling the lake.

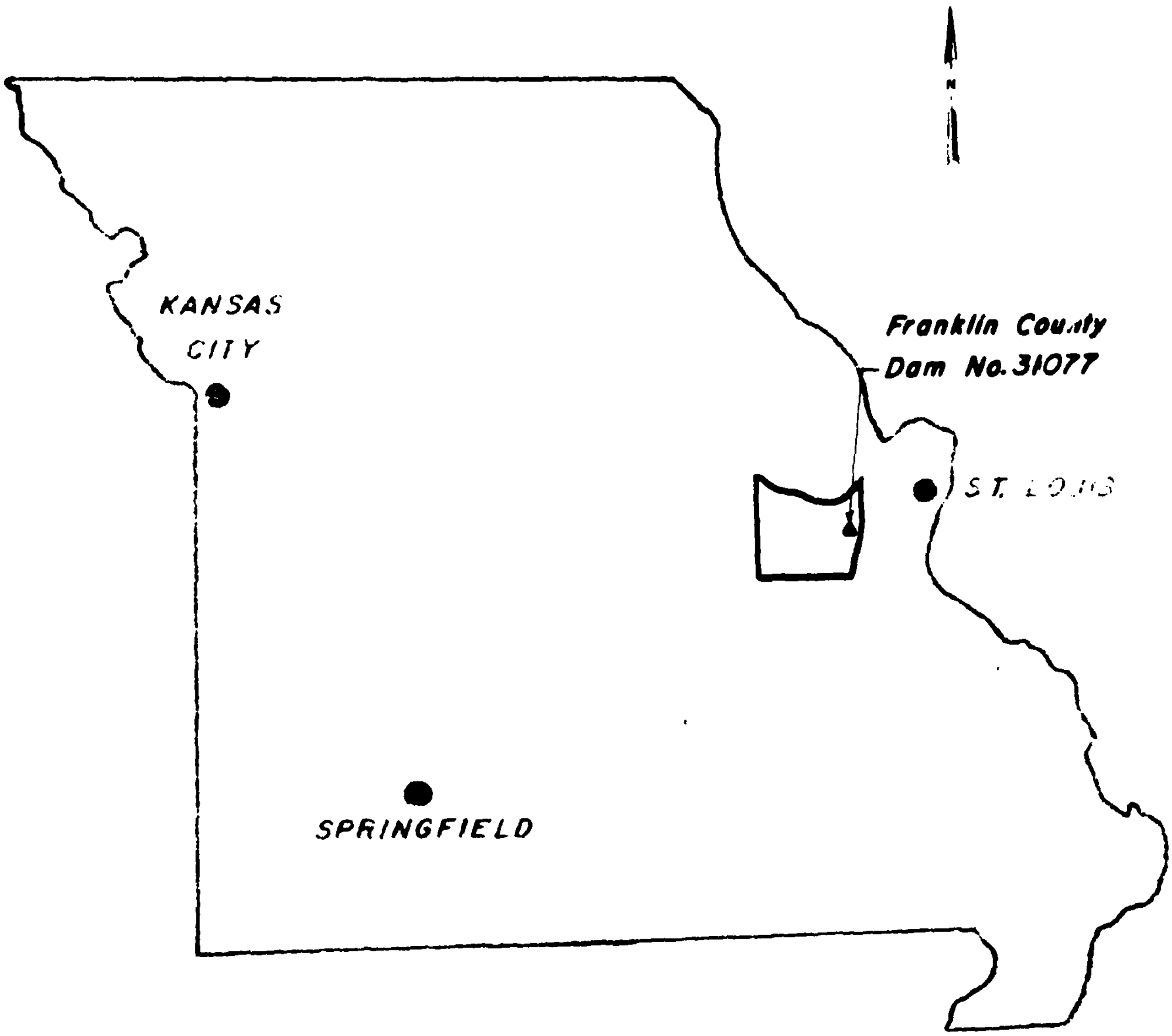
(5) A trash rack or a system designed to prevent trash from clogging the inlet of the pimary spillway pipe should be installed.

(6) After the current work to stop the seepage is completed and the lake fills, the seepage areas should be investigated by an engineer experienced in the design and construction of dams.

(7) A detailed inspection of the dam should be made periodically by an engineer experience in the design and construction of dams.

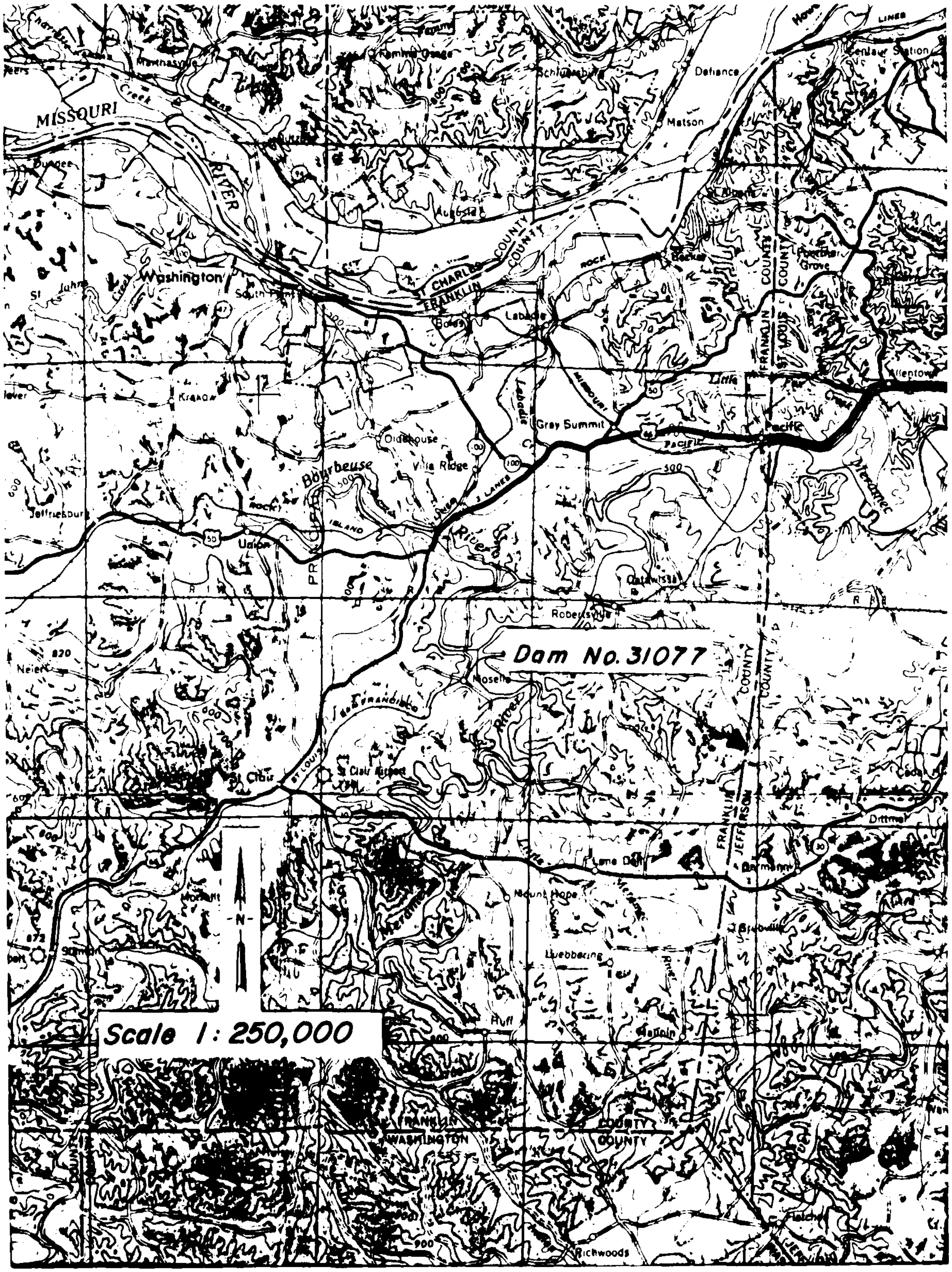
(8) The drain pipe valve should be checked to see if the valve is malfunctioning or if it needs to be closed.

APPENDIX A



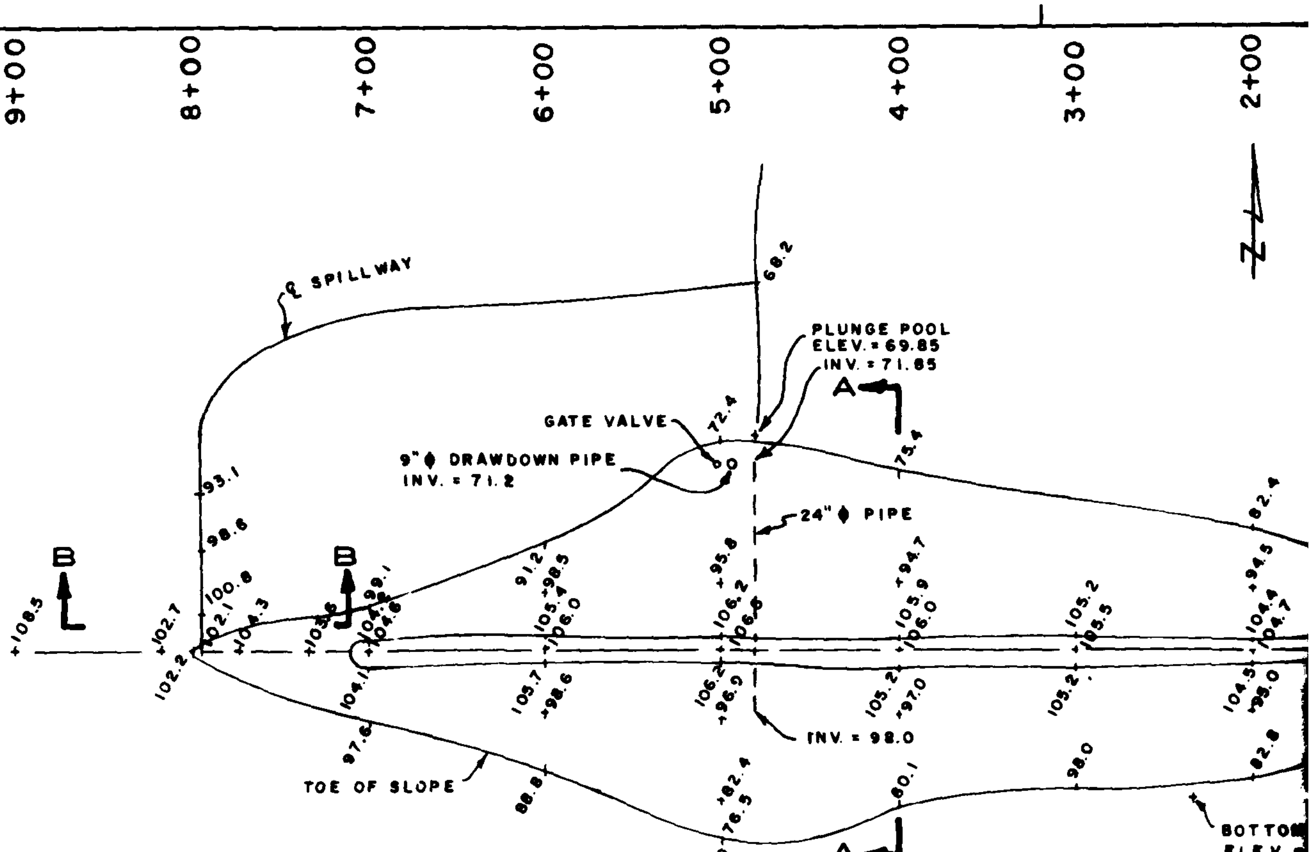
LOCATION MAP

SHEET 1 OF APPENDIX A



SITE VICINITY MAP

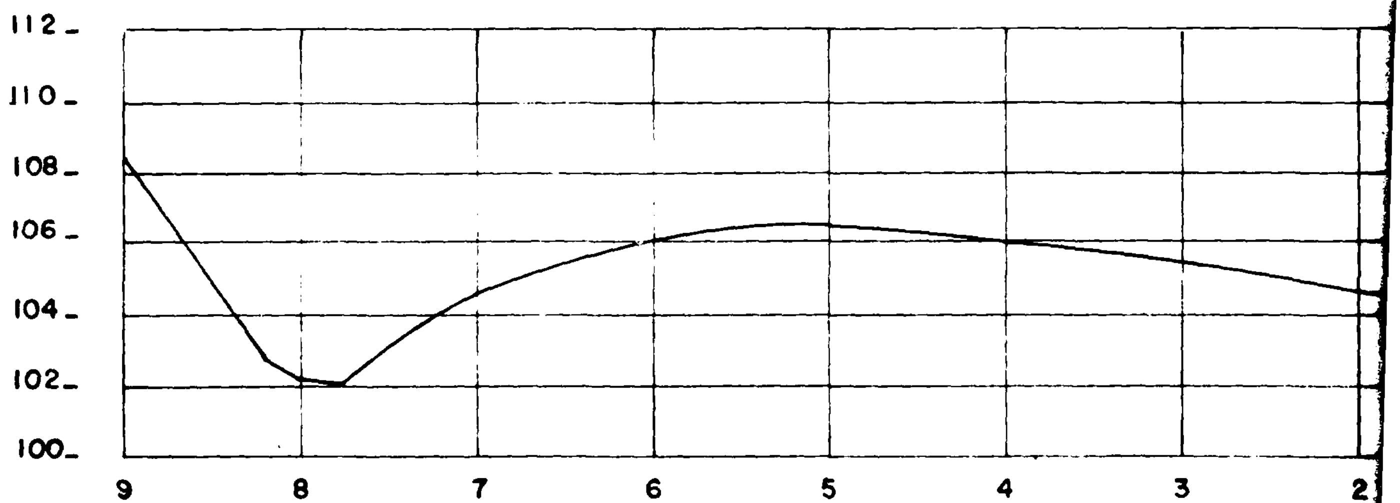
Sheet 2 Appendix A



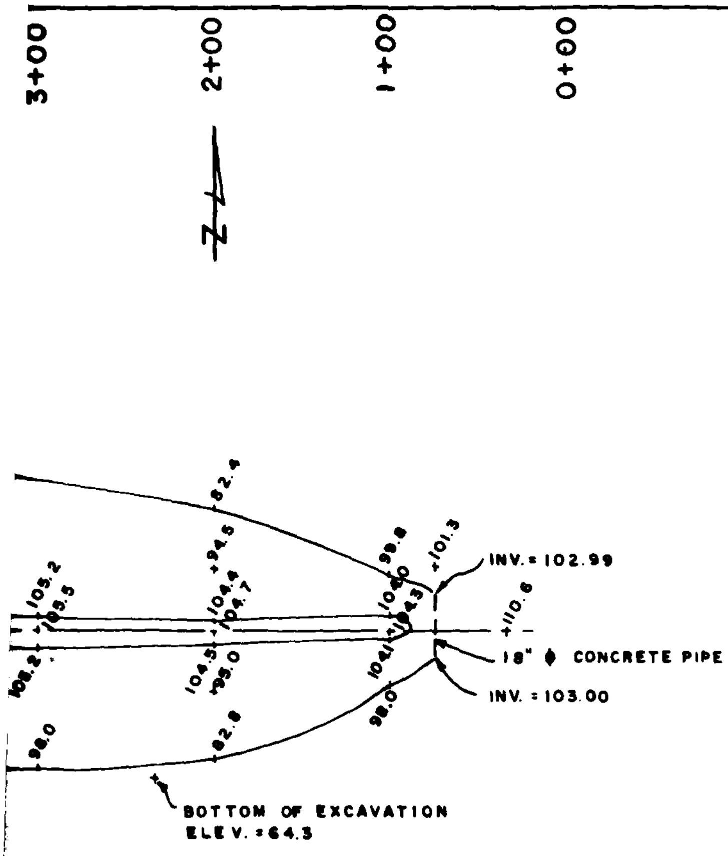
BENCHMARK:
TOP OF 24" ϕ PIPE
STA 4+81 34' LEFT OF ℄
ELEV. = 100.00

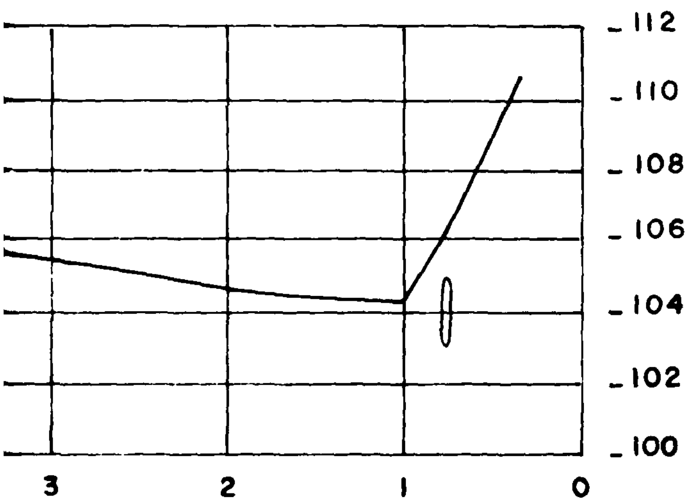
PLAN VIEW

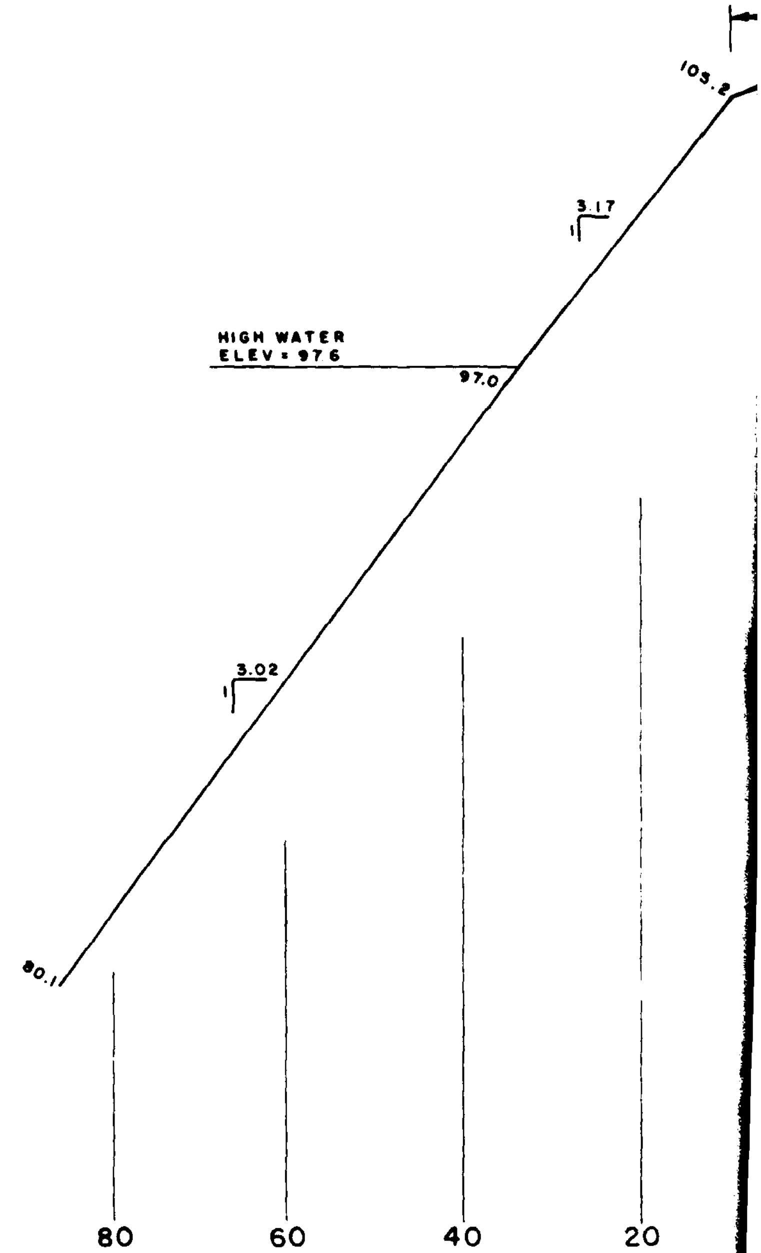
SCALE: 1" = 100'



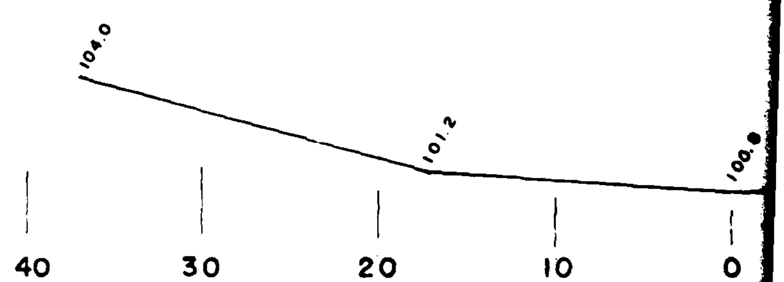
PROFILE



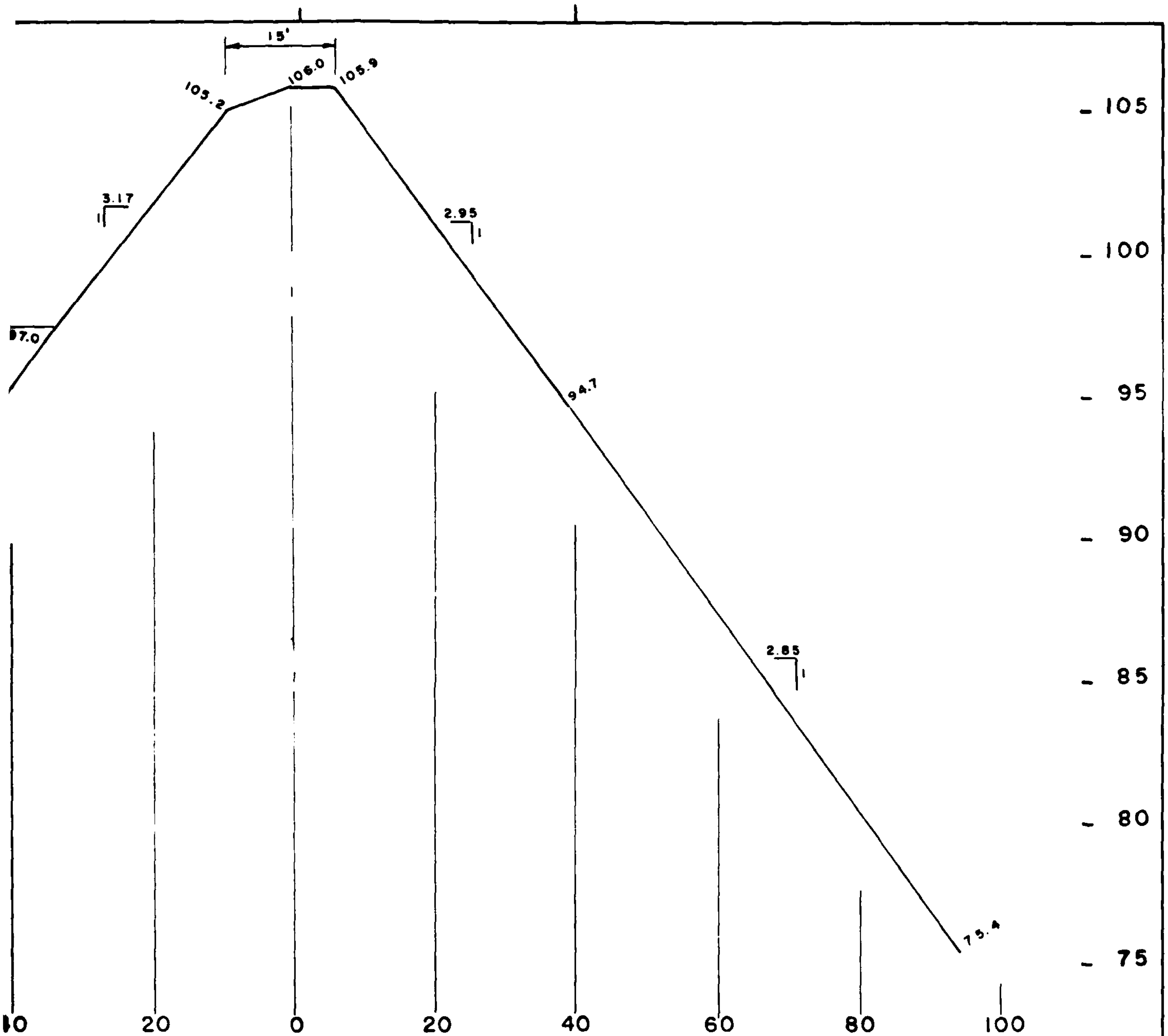




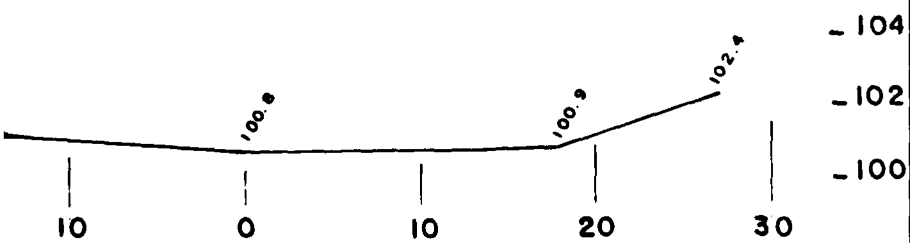
SECTION



SECTION B-B 20' RT.



SECTION A-A STA 4+00



B-B 20' RT. STA 1+97

ANDERSON ENGINEERING, INC.
730 NORTH BENTON AVENUE
SPRINGFIELD, MISSOURI 65802

VON DER AHE DAM

MO. No. 31077

PLAN & PROFILE

FRANKLIN COUNTY, MO.

Sheet 3 of Appendix A

13

x 307, Union, Missouri 63084

November 29, 1976

Mr. Russel Von Der Ahe
600 Rudder
Fenton, Missouri 63026

Dear Mr. Von Der Ahe:

The information in this letter is all estimated from the topo sheets and the figures I had from Bill Tyree.

Based on a 35 foot fill height with a 12" pipe approximately 154 feet long, you will need two feet of height on it witha two foot spillway. This will enable you to raise the water level two feet, as Bill has staked it.

You will need four 48" x 48" anti-seep collars. The first one should be placed 27 feet from the inlet end and 25 feet apart after the first one. One trash rack will also be needed.

Based on the topo map, the estimated pool size is 15 acres. In talking with Bill Tyree this morning, he had figured it about 25 acre pool. With the way the centerline of fill is proposed, I would recommend at least a 4:1 slope on the front to help control the wave wash.

If you need any further information, please contact us.

Sincerely,

Dale Dowdy
Soil Conservation Technician

-40
6/73
e Code: ENG-13

UNITED STATES DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

DESIGN SHEET FOR CLASS ~~II,~~ III, ~~IV~~ * DETENTION STORAGE STRUCTURE
WITH ~~DROP INLET SPILLWAY~~ -- ~~HOOD INLET SPILLWAY~~ -- CANOPY INLET SPILLWAY *

Landowner Russell Van Der Ahe County Franklin

Plan.
Design by D. D. Date 11-24-76 Checked by ____ Date ____

Drainage area = 280 ac. Height x storage = ____ x ____ = ____

WATERSHED CONDITIONS AND FACTORS

Location factor:	L = —
Infiltration factor: (ab~~ove~~) (average) (be~~low~~) *	I = —
Topographic factor: 10 % average slope	T = 1.1
Shape factor: runoff distance = 6000 + ft.	S = —
Cover factor: cropland — %, pasture 20 %, timber 80 %	V = 0.6
Contouring factor:	C = —
Storage factor: — % terraced	P = —

PEAK RATE OF RUNOFF AND VOLUME OF RUNOFF

Product of factors = L x I x T x S x V x C x P = 0.7 Q_{10} = 378 c.f.s.

V x 1 = 0.6 x 1.0 = 0.6

For Principal Spillway Design:

10 -year peak rate of runoff = Q_{ip} = 1.0 x 378 c.f.s. = 378 c.f.s.

Rate of volume of runoff = 0.09 ac. ft./ac. (Table 1, 1519)

Total volume of runoff = V_{rp} = (drainage area) x (rate of volume of runoff) x L =

280 ac. x 0.09 ac. ft./ac. x 1.0 = 25 ac. ft.

For Both Spillways (Total Structure):

25 -year peak rate of runoff = Q_i = 1.3 x 378 c.f.s. = 491 c.f.s.

Rate of volume of runoff = 0.12 ac. ft./ac.

Total volume of runoff = V_r = 280 ac. x 0.12 ac. ft./ac. x 1.0 = 34 ac.ft.

*Mark out those items that do not apply.

Instructions for use of form: Make one pencil copy for applicable structure. File with other worksheets and structure plan in landowner's folder in field office.

SHEET 5 OF APPENDIX A

PRINCIPAL SPILLWAY DESIGN

Available storage at stage of 2 ft. = V_{sp} = 30 ac. ft. (See map)

$V_{sp} \div V_{rp}$ = 30 ac. ft. ÷ 25 ac. ft. = 1.2. $Q_{op} \div Q_{ip}$ = ______ (Table 2, 1519)

Q_{op} = ______ c.f.s. x ______ = ______ c.f.s.

nduit:

Type Smooth Iron Length = 151 ft. Total head on conduit = 3.3 ft.

Diameter = 12 in. Discharge capacity = 8.7 c.f.s. (1520)

Minimum entrance head = 1.6 ft. (1510 or 1511)

$\frac{25 \times 12}{10' \times 8.7} = \frac{300}{6.96} = 43$ hrs.

Riser: **

~~Type~~ ______ Height = ______ ft. Diameter = ______ in. (1511)

EMERGENCY SPILLWAY DESIGN

Control Section:

Depth of flow = 1 ft. V_s at this depth = 45 ac. ft. (See map)

$V_s \div V_r$ = 45 ac. ft. ÷ 34 ac. ft. = 1.3

$Q_{op} \div Q_i$ = 8.7 c.f.s. ÷ 491 c.f.s. = .018. $Q_{oe} \div Q_i$ = ______ (Table 3, 1519)

Q_{oe} = ______ c.f.s. x ______ = ______ c.f.s.

Width = ______ ft. Total depth = depth of flow + freeboard = ______ ft. + 1.0 =

______ ft. Use ______ ft. (Table 4, 1517)

Exit Section:

Slope = ______ % Quality of vegetation: (fair) (good) (excellent) *

(Less) (More) * erosive soils. Permissible velocity = ______ f.p.s. (1517)

Depth = ______ ft. Design velocity = ______ f.p.s. Width = ______ ft. (1517 or 1505)

Use width of ______ ft.

ANTI-SEEP COLLARS

ength of saturated zone = L = ______ ft. Collar addition = ______ ft. (1515)

Number = n = (L x ______) ÷ V = (______ x ______) ÷ ______ = ______. Use ______ collars.

* Mark out those items that do not apply.
** Applies only to Drop Inlet Spillways.

ENGINEERING GEOLOGIC REPORT OF THE VON DER AHE LAKE

FRANKLIN COUNTY, MISSOURI

LOCATION: SE¼, SW¼, Sec. 24, T. 4_ N., R. 2 E., Lonedell Quadrangle.

The existing lake built approximately 1 year ago is in dolomite of the Jefferson City formation. The dolomite of the Jefferson City is relatively impermeable vertically but will transmit considerable quantities of water horizontally in the upper 1-2 feet of bedrock where extensive weathering has taken place. In this area, a sandstone bed is present that can vary from 2-4 feet in thickness and is observable on the lower valley wall just downstream of the left abutment. This sandstone bed will be present on the right abutment also but is covered by colluvial soil.

Leakage under or around the lower left abutment is causing water to move at a shallow depth below the surface soil to emerge as springs and seeps in a field downstream of the dam on the adjoining property. When the lake is full, the seeps and springs are reported to spout considerable quantities of water but lowering of the lake resulted in a dry condition on the date of this investigation. Evidence, however, suggests that substantial quantities of water surfaced on the downstream land in the floodplain area.

The depth of core or condition of bedrock in the core trench or the method of placement of the core during original construction is not known. If no bedrock was removed across the valley bottom during the initial construction, it is probable that leakage is through the upper 1-2 feet of bedrock immediately below the core area. If a positive cutoff was accomplished during initial construction, then it is speculated that leakage is horizontally around the lower right abutment through the sandstone layer.

The surface soils on the floodplain downstream of the dam are of such a quality that this water can't surface and is being transmitted at a shallow depth below the alluvial-colluvial soil. A weakness in the soil cap on the downstream property allows the water to surface in substantial quantities.

This water can in all probability be intercepted upstream of the property by a shallow trench with diversion to the creek. This interceptor ditch could be backfilled with a coarse grained material to act as an underdrain or a perforated pipe could be installed that would solve the immediate problem of the downstream landowner.

To repair the leak at the dam, it would be necessary to determine the area of leakage and then recore the lower right abutment on the upstream toe of the dam. To test for the probable location of leakage, it is recommended that a backhoe be employed to dig a shallow trench on the downstream toe of the dam starting on the east side of the stream bed and progressing to the east until water is intercepted. Then the trench should proceed farther east until no more water is intercepted. This method would outline the area of shallow leakage under the dam. Once this has been determined, it would be a matter of recoring the dam on the lake side to a depth and a lateral dimension equal to the water loss zones located in the downstream side in the backhoe trench.

Removal of dirt material from the valley walls upstream of the dam for construction of the dam allows lake water to come into contact with the bedrock in numerous places. Horizontal movement of water in the shallow bedrock is normal and unless the core was seated in sound rock, leakage is probable. For this reason, it is felt that the leakage is very shallow and at the contact between the bottom of the core and the bedrock or at the sandstone layer in the lower left abutment.

Thomas J. Dean

Thomas J. Dean, Geologist
Engineering Geology Section
Geology & Land Survey
September 20, 1978

orig: Leonard Knoernschild
SCS, Union

See addendums to this report

ADDENDUM TO THE VON DER AHE LAKE SITE

FRANKLIN COUNTY, MISSOURI

A trench excavated on the downstream tow of the dam from the spillway pipe eastward to the right or east abutment penetrated some 10 to approximately 14' of residual and alluvial material to bedrock. Only small quantities of water were present in the exploration trench on the date of this investigation (9-26-78). The elevation of the lake level in reference to the trench is not known.

Several vertical joints may be present in the dolomite in the bottom of the trench that are able to transmit water down the valley. Wooden sticks were stuck in areas that may be potential leakage points relative to vertical cracks in the rock. These two areas should be excavated with a shovel or other suitable tools to see if the joints go to any depth and if they are open to the point where water from the bottom of the lake can move through them.

In addition, the small vertical bluff near the ~~left~~ right abutment that is exposed as a ledge should be cleaned off to determine if openings exist underneath this 1-2 foot thick ledge. This horizontal opening of open bedding plane could easily be a point of transmission of water around the end of the lower abutment when sufficient quantities of water is in the lake.

In addition, the sandstone ledge thought to exist in this area is exposed just above the small vertical limestone ledge. Not enough excavation above the limestone ledge was accomplished to determine the full thickness of the sandstone bed. It is recommended that more earth be removed from the ~~left~~ right abutment from the top of the limestone ledge to fully expose the sandstone and removal of dirt from that slope of the trench be continued until bedrock is exposed all the way to the elevation of the water line.

The type of leakage involved could very well be a vertical joint such as those exposed in the very bottom of the trench or it could easily be horizontally moving water at the base of the limestone ledge or through the sandstone immediately above the limestone ledge. Exposures of these various points of leakage with equipment and/or hand tools to where good observation of them can be done may well reflect probable leakage points. The vertical joints (assuming that they are open vertical joints once they have been cleaned out) could probably be filled with cement at the point of exposure downstream of the dam prior to refilling of the trench. These same joints then should be exposed on the upstream side of the dam and likewise filled with cement in that area to prevent water from getting under the dam.

If the problem is determined to be open bedding planes below the limestone ledge or in the sandstone above the limestone ledge, then the problem must be entirely corrected on the lake side of the dam. This can be done by excavation of the soil material to that level, and then excavating the bedrock back into the hill a sufficient distance to where it becomes solid and/or the bedding planes underneath the sandstone or the limestone pinch out.

If the point of water loss cannot be determined by the above recommendations, then probing of the bedrock in the bottom of the trench with a jackhammer or small drill would be recommended. Much of the rock exposed in the bottom of the core trench could be considered "false" bedrock in that the upper 1 foot to 18 inches of rock can be expected to be capable of transmitting water. This is due to extensive weathering of the upper layers of rock in past geologic time. Holes could be drilled or probed to a depth of 3 to 4 feet every 5 feet or so that may well expose open channels immediately below the surface layer.

In summary, additional work by hand and equipment is necessary to better expose the rock in the core in the exploration trench. Lack of water in the lake has produced a hydraulic condition where the point of water loss is not obvious. The physical condition of rock exposed in the core trench suggests that numerous points of water loss are probable but probably can be corrected using normal construction practices.

Thomas J. Dean

Thomas J. Dean, Geologist
Engineering Geology Section
Geology & Land Survey
October 4, 1978

orig: Leonard Knoernschild
SCS, Union

AN ADDENDUM TO THE VON DER AHE LAKE

FRANKLIN COUNTY, MISSOURI

<u>LOCATION:</u> S½, NW¼, NE¼, NW¼, Sec. 25, T. 42 N., R. 2 E., Lonedell Quadrangle.

On 1 December 1978, in the company of the SCS, Union, Mo., an attempt was made re-evaluate leakage conditions at the Von Der Ahe #2 lake at the above location. The lake was at a very low level due to drawdown in a drain pipe under the dam. A test pit excavated on the upstream side of the dam was exposed with the lake level 3 to 4 feet above the water level in the test pit. Several days prior to the first of December, water from the lake was observed flowing into the test pit when the lake level and the test pit water level were roughly the same. The downstream test pit had a large quantity of water in it so it could not be determined whether lateral flow around the dam was taking place between the upstream pit and the downstream pit. The upstream test pit did reveal, however, that an excess of 6 feet of clayey soil overlies much of the bedrock in the lake basin with soil removed to bedrock starting at a point about halfway to the water line. It is thought at this time that the relatively thick blanket of clayey material in the lake bottom would preclude rapid vertical water movement into the bedrock under the soil blanket. For this reason, padding of the lake bottom is not thought necessary.

Water loss in the test pit took place because leakage zones under the natural clay blanket were exposed and probably does not represent the general leakage problem in the lake.

From discussions at the site, it is felt that the following tests should be made to determine probable areas of leakage so that recommendations on how to correct these leakage problems can be made.

1. Close off the valve and allow the lake to fill to a sufficient level where leakage takes place. It is thought at this time that the leakage will begin to take place when the lake is more than half full. Prior to lake filling, it is recommended that the test pit on the lake side of the dam be filled with compacted clayey material in 6 inch lifts to eliminate this local leakage point.

The second recommendation is that when the lake level approaches that point where lake water comes in contact with the exposed bedrock on the east side that the test pit previously excavated downstream of the dam be cleaned out by removal of mud and water. Soil material in the trench should be removed all the way to bedrock coming up the east valley wall so bedrock is exposed the full length of the trench on the east side. If horizontal movement of water takes place around the dam, the water should move rapidly into the trench and the point of entry noted. It is thought that during construction of the original downstream exploration trench, no water was noted because the lake level was lower than the leakage point in the lake.

If water is observed coming into the downstream trench as the lake fills, then that existing level of the lake can be assumed to be the leakage point, and grouting or recoring on the upstream toe of the dam can be accomplished.

In the event that no water comes into the downstream trench as the lake approaches full pool, then it can be assumed that leakage is in the bedrock in a confined condition with emergence in the field downstream of the property line. In this event, then probing with a jackhammer or air track type drill in the bedrock in the downstream trench would be necessary to pinpoint the subsurface movement of the water. Once the point of water loss is detected on the downstream side of the dam, then drilling and grouting through the crown of the dam would be necessary to cut off this leakage point.

In summary, the following steps are recommended to determine the point of water loss from the lake.

1. Fill in the upstream exploration trench to exclude lake water from it.

2. Shut off the valve and allow the lake to fill.

3. When the lake is at least ½ full, clean out the exploration trench on the downstream side of the dam in an attempt to observe inflow of water. The inflow would be expected at a relatively high stage of the lake level.

4. If when the lake is approximately full and no excessive leakage into the downstream trench is observed, then probing of the bedrock in the trench bottom by drilling means would be necessary.

Thomas J. Dean, Geologist
Engineering Geology Section
Geology & Land Survey
December 7, 1978

orig: Leonard Knoernschild
SCS, Union

,07, Union, Missouri 63084

December 11, 1978

Mr. Russell L. Von Der Ahe
600 Rudder
Fenton, Missouri 63026

Dear Mr. Von Der Ahe:

Enclosed is additional information from the Missouri Department of Geology written by Tom Dean. He asked me to send this material on to you.

In reviewing what was discussed at the lake site and what is included in this report, we would recommend the following:

First, close up the test pit on the upper side of the dam by replacing with clayey material in 6 inchlayers in the pit and making sure this is walked in in thin layers with the machinery so that the material is well compacted.

Second, close the valve on the draw down tube and allow the lake to fill. When the water level reaches 16 to 18 feet, be sure and let us know so that additional observations can be made. I suggest you contact Tom Dean at the same time you let us know. But, if you prefer, we will call Mr. Dean for you.

I am sending a copy of this letter and the enclosed material to your caretaker, Gene Pursley.

If you have any questions about this or if we can be of further help, please let us know.

Sincerely,

Leonard Knoernschild
District Conservationist

Enc.

cc: Gene Pursley

7

ADDENDUM TO THE VON DER AHE LAKE SITE

FRANKLIN COUNTY, MISSOURI

On 4-2-79 in the company of Mr. Bill Tyree, Mr. Von Der Ahe, and the SCS, an additional discussion at the dam site was made. Excavation near the base of the right abutment to a depth of about 10 feet encountered an open bedding plane in the underlying dolomite. This bedding plane is reported to take considerable quantities of water and may well be the opening that reported water loss from the lake is taking to emerge at a spring downstream of the Von Der Ahe property. The fact that the open bedding plane is covered by large quantities of clayey soil leads me to believe that water must back up over the bedrock in the right abutment and then move essentially vertical at the soil bedrock contact down to the open bedding plane, if in fact this is the outlet for the water. When the trench downstream was excavated, this same ledge was uncovered and no water emerged at this same bedding plane area. At the time, the lake was at least ½ or 2/3 full. Anyway, the recommendation to the contractor was to recore the right abutment on the upstream toe of the dam to include the open bedding plane in the vicinity of the excavated hole.

About ¼ pound of dye was placed in the water in the hole and the emergence point downstream bugged in an attempt to verify that water moving through this open bedding plane does emerge at the spring. The bugs will be picked up on Thursday, the 5th of April.

Thomas J. Dean, Geologist
Engineering Geology Section
Geology & Land Survey
April 4, 1979

MISSOURI DEPARTMENT OF NATURAL RESOURCES
P.O. Box 250 · Rolla, Missouri 65401 (314) 364-1752

April 16, 1979

Bill Tyree
TyRee Excavating Co.
Rural Route 1
Sullivan, MO 63080

Dear Bill:

The dye that I placed in the excavated pit just upstream of the right abutment of the Von Der Ahe lake came through the system and was recovered at the spring downstream on Thursday, the 5th of April.

I suspect that any leakage that takes place on that whole valley wall (right abutment) would tend to come out at that point.

I would recommend that the re-coring operation start or finish about at the location of that large bedding plane opened up at base of the pit. I would recommend that the re-coring operation take place from that point all the way to the water line. I suspect there is numerous points where the water can more around the right abutment so re-coring into rock a sufficient depth where rock is good and solid would be advisable along that whole stretch of area.

In summary, the dye placed in the pit moved under the dam and emerged at the spring downstream. It is recommended that re-coring of the dam progress from that pit all the way to the shoreline to the east.

Yours truly,

Thomas J. Dean

Thomas J. Dean, Geologist
Engineering Geology Section
Geology & Land Survey

cc: SCS, Union

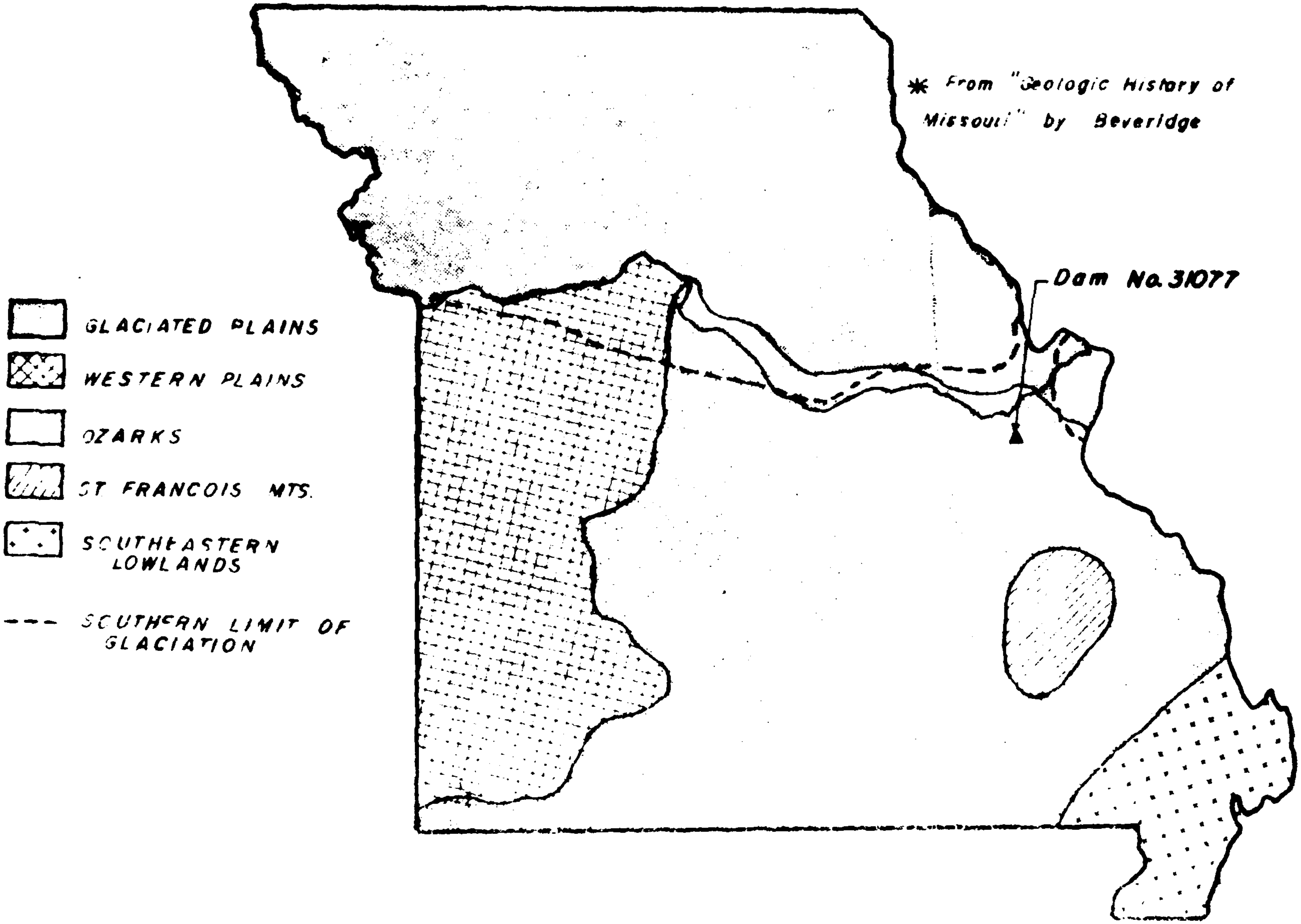
SHEET 15 OF APPENDIX A

Joseph P. Teasdale Governor
Fred A. Lafser Director

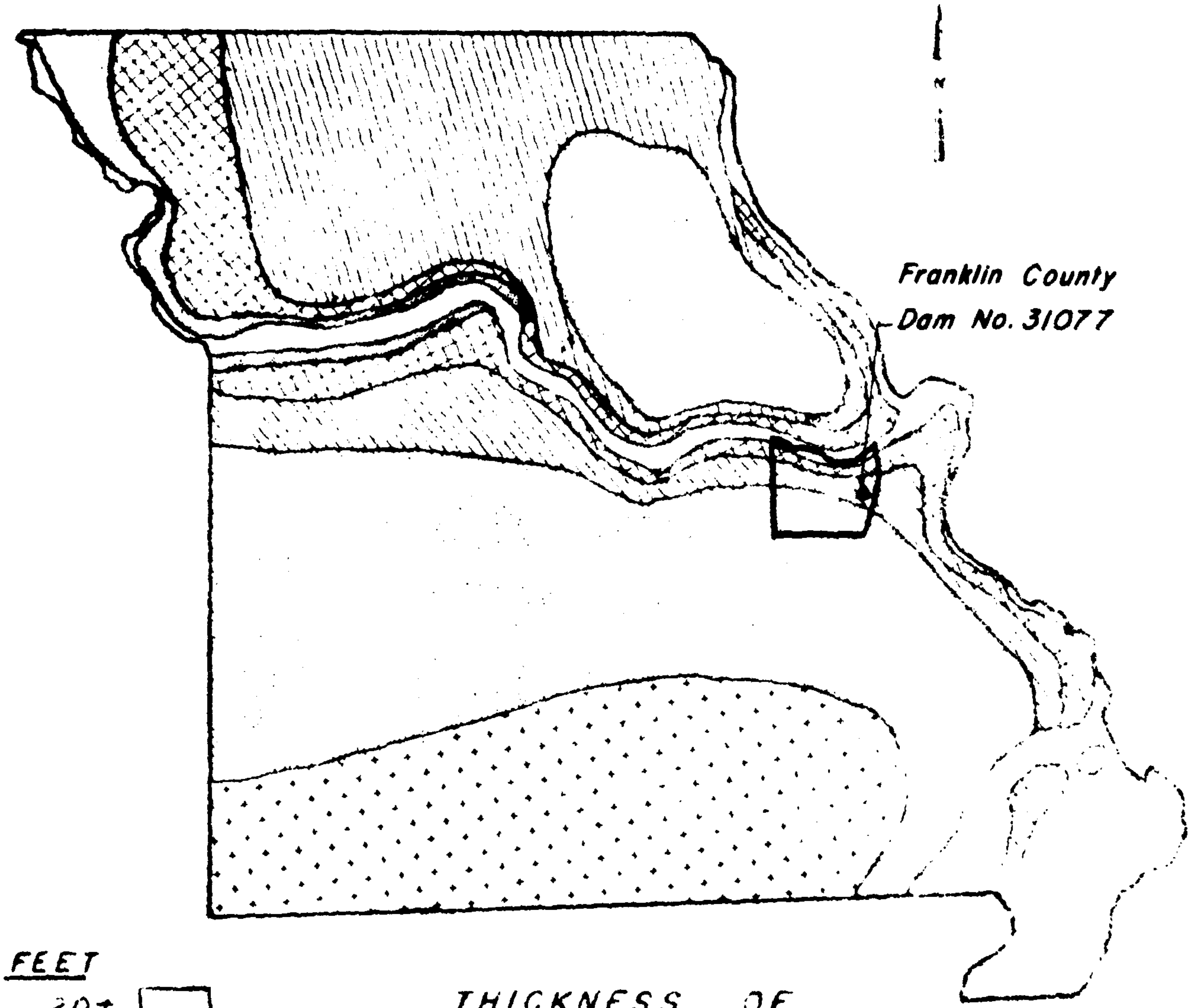
Division of Geology and Land Survey
Wallace B. Howe Director

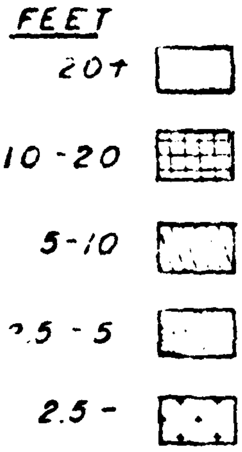
APPENDIX B

MAJOR GEOLOGIC REGIONS OF MISSOURI



* From "Soils of Missouri"

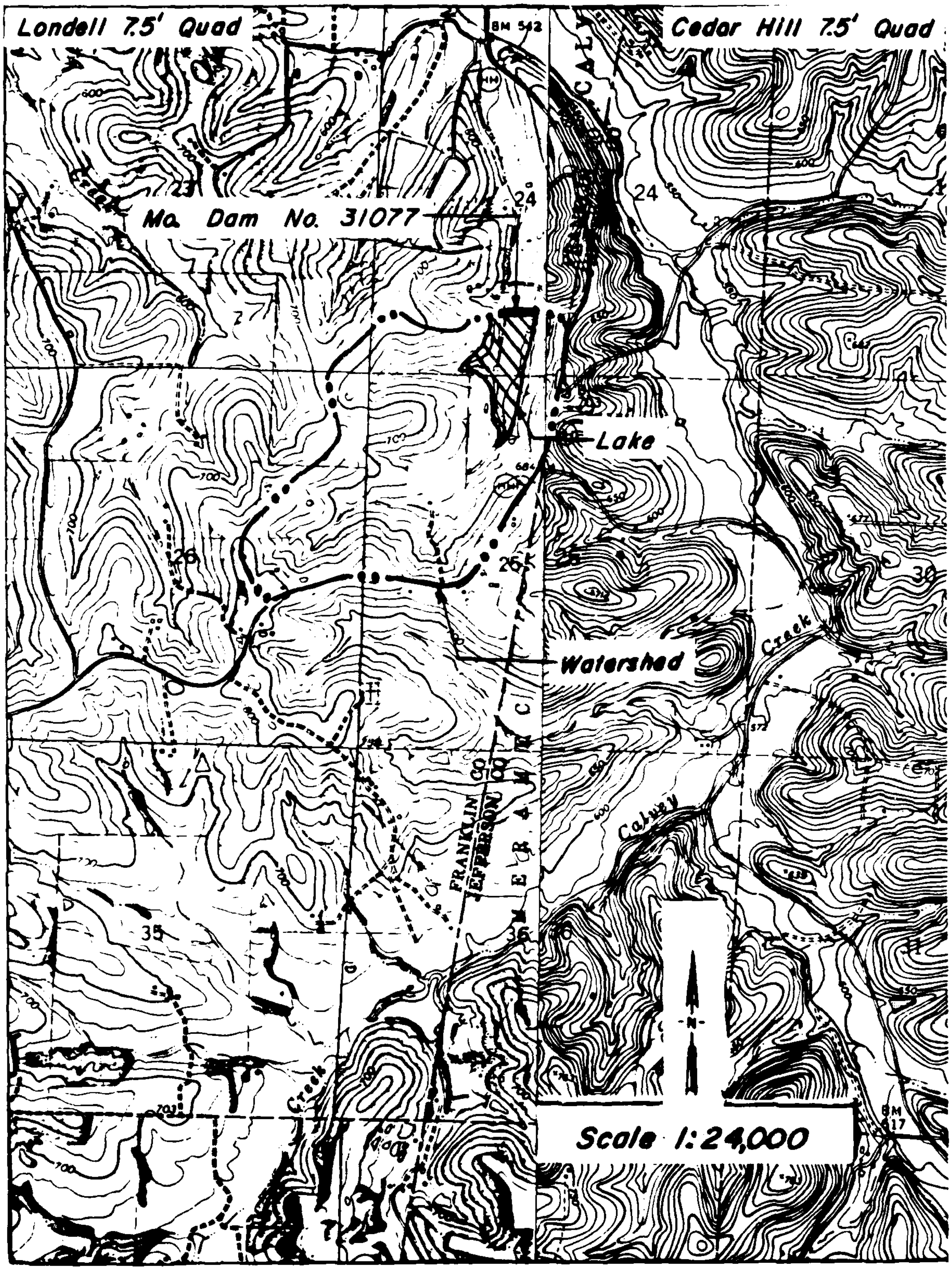




THICKNESS OF LOESSIAL DEPOSITS

SHEET 2 OF APPENDIX B

APPENDIX C



LAKE AND WATERSHED MAP

Sheet I Appendix C

HYDRAULICS AND HYDROLOGIC DATA

Design Data: From Field Measurements and Computations

Experience Data: No records are available. The owners caretaker, Mr. Persley, indicated that the dam has never been overtopped and that the primary spillway, but not the emergency spillway, has operated in the past. Wave erosion of the upstream face of the dam indicates that water level has been above the invert elevation of the primary spillway.

Visual Inspection: At the time of inspection, on 25 June 1979, the pool was empty.

Overtopping Potential: Flood routings were performed to determine the overtopping potential. The watershed and the reservoir surface areas were obtained by a planimeter-from the U.S.G.S. Londell and Cedar Hill , Missouri 7.5 minute quadrangle maps. The storage volume was developed from this data. A 5 minute interval unit graph was developed for this watershed, which resulted in a peak inflow of 1012 c.f.s. and a time to peak of 13 minutes. Application of the probable maximum precipitation minus losses results in a flood hydrograph peak inflow of 5374 c.f.s. Rainfall distribution for the 24 hour storm was according to EM 11102-1411.

Based on our analyses, the combined spillways will pass 33 percent of the Probable Maximum Flood (PMF). The Probable Maximum Flood is defined as the flood discharge that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. The recommended guidelines from the Department of the Army, Office of the Chief of Engineers, require that the structure (small size with high downstream hazard potential) pass 50 to 100 percent of the PMF, without overtopping. Considering the volume of water impounded, the height of the dam and the large floodplain downstream, 50 percent of the PMF has been determined to be the appropriate spillway design flood.

The routing of 50 percent of the PMF through the spillway and dam indicates that the dam will be overtopped by 0.98 ft. at elevation 105.28. The duration of the overtopping will be 1.17 hours, and the maximum outflow will be 1817 c.f.s. The maximum discharge capacity of the combined spillways is 701 c.f.s. Analysis of the data indicates that the 100-year frequency flood will not overtop the dam. The computer input, output and hydrographs for 50 percent of the PMF are presented on Sheets 5, 6 & 7 of Appendix C.

OVERTOPPING ANALYSIS FOR VON DER AHE DAM

INPUT PARAMETERS

1. Unit Hydrograph - SCS Dimensionless - Flood Hydrograph Package (HEC-1); Dam Safety Version Was Used.
   Hydraulic Inputs Are as Follows:

   a. Twenty-four Hour Rainfall of 25.5 Inches for 200 Square Miles - All Season Envelope

   b. Drainage Area = 292 Acres; = 0.46 Square Miles

   c. Travel Time of Runoff 0.30 Hrs.; Lag Time 0.18 Hrs.

   d. Soil Conservation Service Soil Group C

   e. Soil Conservation Service Runoff Curve No. 85 (AMC III)

   f. Proportion of Drainage Basin Impervious 0.07

2. Spillways

   a. Primary Spillway: 24 inch I.D. steel pipe, crest Elevation 98.0

   b. Emergency Spillway:

      (1) Trapedzoidal cut on natural ground, crest elev. 102.2
      Length 30 ft.; Side Slopes Vary; C = Varies

      (2) 18 inch I.D. Concrete pipe, crest elev. 103.00

   c. Dam Overflow

      Length ---- Ft.; Crest Elev. 104.3; C = Varies

3. Spillway and Dam Rating:

   Curve Prepared by Hanson Engineers. Data Provided To Computer on Y4 and Y5 Cards for the spillways and on $6 and $V cards for the dam (See Sheet 8, Appendix C).

   Formula and method used:

   Primary Spillway: Charts for steel pipe with entrance control.

Earth cut emergency spillway and dam:

$$\frac{Q^2}{g} = \frac{A^3}{T}$$

Concrete pipe emergency spillway: Charts for concrete pipe with entrance control.

Note: Time of Concentration From Equation $Tc = \left[\frac{11.9\ L^3}{H}\right]^{.385}$

California Culvert Practice, California Highways and Public Works, Sept. 1942.

SUMMARY OF DAM SAFETY ANALYSIS

1. Unit Hydrograph

   a. Peak - 1012 c.f.s.

   b. Time to Peak 13 Min.

2. Flood Routings Were Computed by the Modified Puls Method

   a. Peak Inflow
      50% PMF 2687 c.f.s.; 100% PMF 5374 c.f.s.

   b. Peak Elevation

      50% PMF 105.28; 100% PMF 106.38

   c. Portion of PMF That Will Reach Top of Dam

      33%; Top of Dam Elev. 104.3 Ft. (Lowest Point)

3. Computer Input and Output Data are shown on the following sheets of this Appendix.

```
A       OVERTOPPING ANALYSIS FOR VON DER AHE DAM ( # 26 )
A       STATE ID NO. 31077  CO. NO. 071  CO. NAME FRANKLIN
A       HANSON ENGINEERS INC. DAM SAFETY INSPECTION JOB # 79511
B     300               5
B1      5
J       1       7       1
J1    .10     .20     .30     .40     .50     .75     1.0
K       0       1                                 3       1
K1      INFLOW HYDROGRAPH COMPUTATION
M       1       2    0.46            0.46       1                       1
P       0    25.5     102     120     130
T                                                -1     -85              0.07
W2   0.30    0.18
X       0     -.1       2
K       1       2                       0       4       1
K1      RESERVOIR ROUTING BY MODIFIED PULS AT DAM SITE
Y                               1       1
Y1      1                                     186.7      -1
Y4     98     100   102.2     103   104.3     105     106   106.5     107
Y5      0      15      28     130     709    1511    4081    6235    9785
$A      0      20      25      27      32
$E     70      98   104.3     106     118
$$     98
$D  104.3
K      99
```

P.M.F. INPUT DATA

SHEET 5 APPENDIX C

********** ********** ********** ********** **********

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
AREA IN SQUARE MILES (SQUARE KILOMETERS)

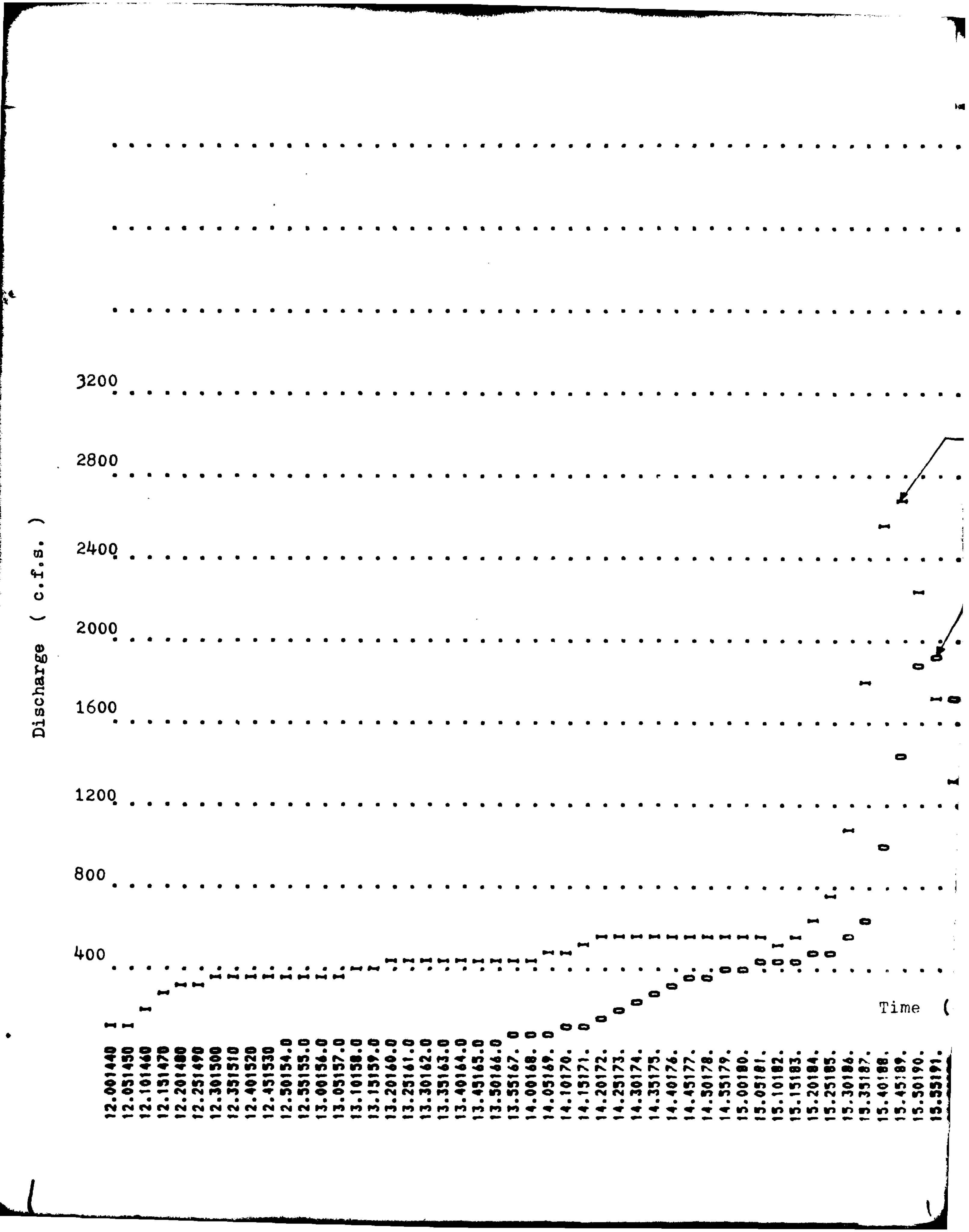
OPERATION	STATION	AREA	PLAN	RATIO 1 0.10	RATIO 2 0.20	RATIO 3 0.30	RATIO 4 0.40	RATIO 5 0.50	RATIO 6 0.75	RATIO 7 1.00
				RATIOS APPLIED TO FLOWS						
HYDROGRAPH AT	1	0.46	1	537.	1075.	1612.	2150.	2687.	4031.	5374.
	(	1.19)	(	15.22)(	30.44)(	45.65)(	60.87)(	76.09)(	114.14)(	152.18)(
ROUTED TO	2	0.46	1	22.	172.	619.	1275.	1930.	3375.	4741.
	(	1.19)	(	0.62)(	4.88)(	17.51)(	36.11)(	54.64)(	95.58)(	134.25)(

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	98.00	98.00	104.30
STORAGE	187.	187.	328.
OUTFLOW	0.	0.	709.

RATIO OF PMF	MAXIMUM RESERVOIR U.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
0.10	101.14	0.00	253.	22.	0.00	19.25	0.00
0.20	103.09	0.00	299.	172.	0.00	17.50	0.00
0.30	104.10	0.00	323.	619.	0.00	16.08	0.00
0.40	104.79	0.49	341.	1275.	0.75	15.92	0.00
0.50	105.16	0.86	350.	1930.	1.00	15.92	0.00
0.75	105.73	1.43	365.	3375.	2.83	15.83	0.00
1.00	106.15	1.85	376.	4741.	4.58	15.83	0.00


Discharge (c.f.s.)
3200
2800
2400
2000
1600
1200
800
400
Time (
12.001440
12.051450
12.101460
12.151470
12.201480
12.251490
12.301500
12.351510
12.401520
12.451530
12.50154.0
12.55155.0
13.00156.0
13.05157.0
13.10158.0
13.15159.0
13.20160.0
13.25161.0
13.30162.0
13.35163.0
13.40164.0
13.45165.0
13.50166.0
13.55167.
14.00168.
14.05169.
14.10170.
14.15171.
14.20172.
14.25173.
14.30174.
14.35175.
14.40176.
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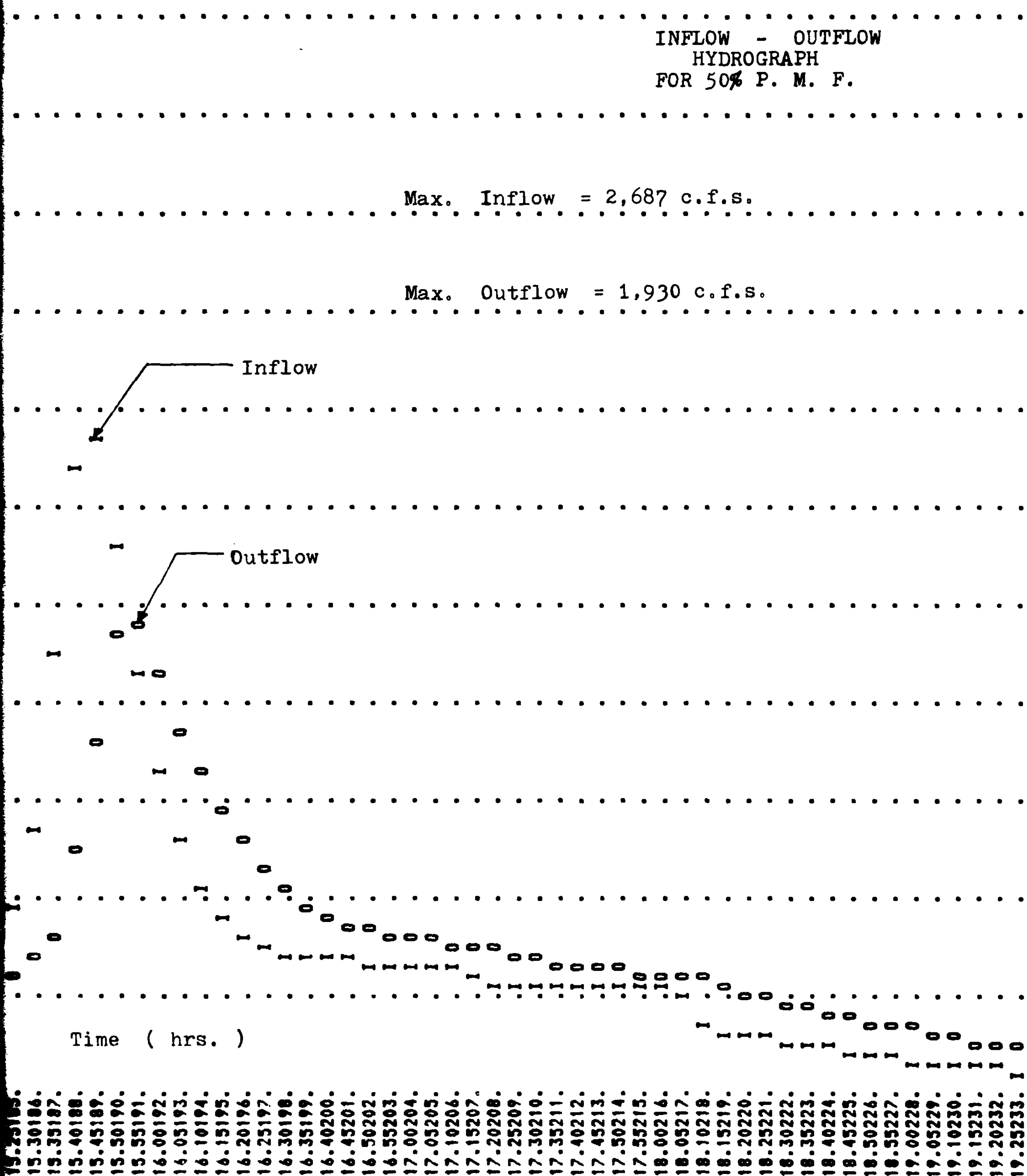

INFLOW - OUTFLOW
HYDROGRAPH
FOR 50% P. M. F.
Max. Inflow = 2,687 c.f.s.
Max. Outflow = 1,930 c.f.s.
Inflow
Outflow
Time (hrs.)

2

```
A       OVERTOPPING ANALYSIS FOR VON DER AHE DAM ( # 26 )
A       STATE ID NO. 31077  CO. NO. 071  CO. NAME FRANKLIN
A       HANSON ENGINEERS INC. DAM SAFETY INSPECTION JOB # 79511
B     300               5
B1      5
J       1       7       1
J1    .10     .20     .30     .40     .50     .75     1.0
K       0       1                             3       1
K1      INFLOW HYDROGRAPH COMPUTATION
M       1       2    0.46            0.46       1                       1
P       0    25.5     102     120     130
T                                                    -1     -85            0.07
W2   0.30    0.18
X       0     -.1       2
K       1       2                       0       4       1
K1      RESERVOIR ROUTING BY MODIFIED PULS AT DAM SITE
Y                               1       1
Y1      1                                         186.7      -1
Y4     98     100   102.2     103   104.3     105   105.5     106   106.5     107
Y5      0      15      28     111     701    1247    1750    2314    2926    3660
$A      0      20      25      27      32
$E     70      98   104.3     106     118
$$     98
$D  104.3
$L      0     177     282     423     630     634
$V  104.3   105.0   105.5   106.0   106.5   107.0
K      99
```

P.M.F. INPUT DATA
Sheet 8, Appendix C

********** ********** ********** ********** **********

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
AREA IN SQUARE MILES (SQUARE KILOMETERS)

OPERATION	STATION	AREA	PLAN	RATIOS APPLIED TO FLOWS RATIO 1 0.10	RATIO 2 0.20	RATIO 3 0.30	RATIO 4 0.40	RATIO 5 0.50	RATIO 6 0.75	RATIO 7 1.00
HYDROGRAPH AT	1	0.46	1	537.	1075.	1612.	2150.	2687.	4031.	5374.
	(	1.19)	(	15.22)(	30.44)(	45.65)(	60.87)(	76.09)(	114.14)(	152.18)(
ROUTED TO	2	0.46	1	22.	172.	617.	1206.	1817.	3252.	4657.
	(	1.19)	(	0.62)(	4.86)(	17.46)(	34.16)(	51.44)(	92.09)(	131.86)(

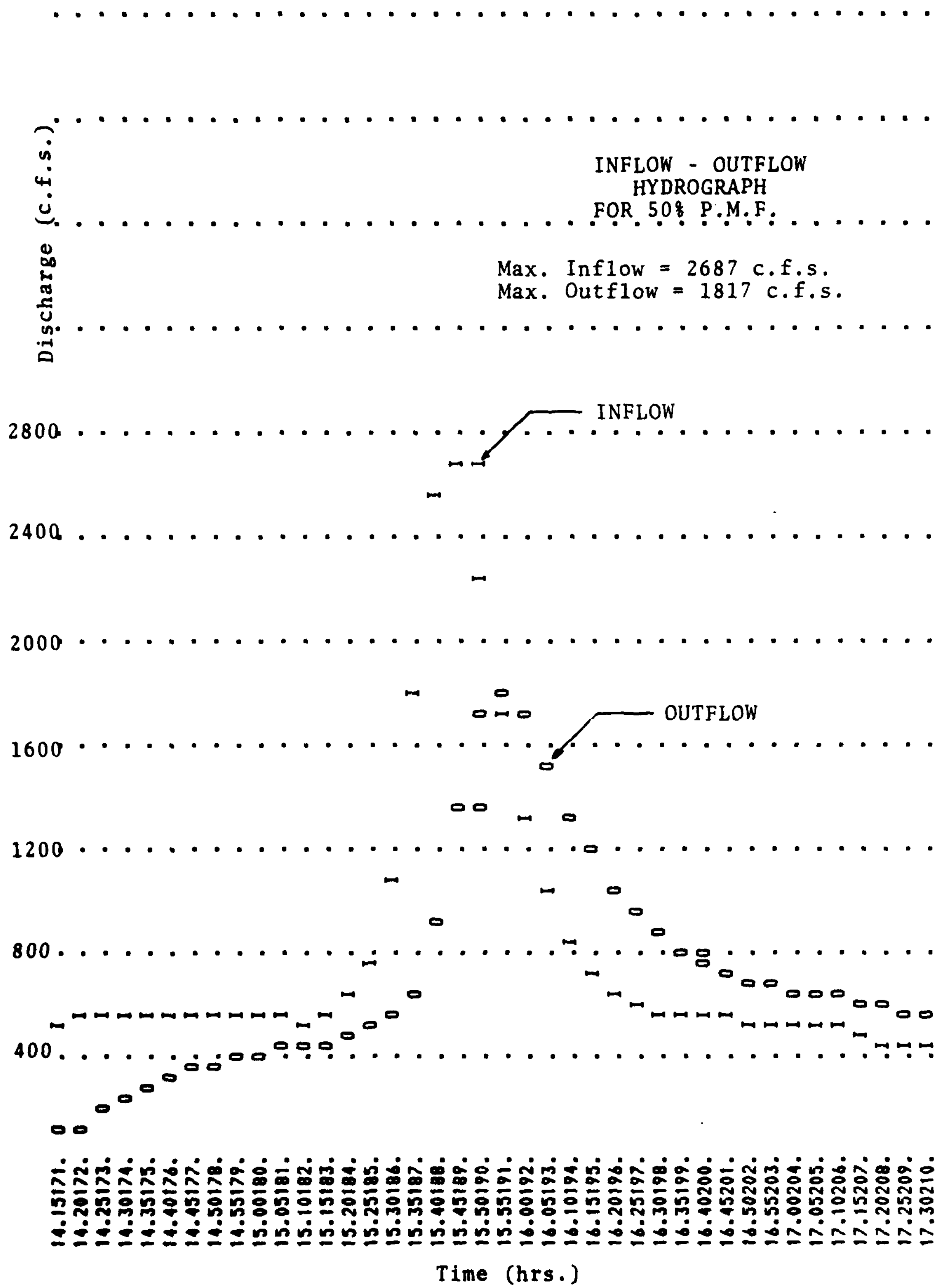
SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	98.00	98.00	104.30
STORAGE	187.	187.	328.
OUTFLOW	0.	0.	701.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
0.10	101.14	0.00	253.	22.	0.00	19.25	0.00
0.20	103.13	0.00	300.	172.	0.00	17.50	0.00
0.30	104.11	0.00	323.	617.	0.00	16.08	0.00
0.40	104.86	0.56	342.	1206.	0.83	16.00	0.00
0.50	105.28	0.98	353.	1817.	1.17	15.92	0.00
0.75	105.93	1.63	370.	3252.	3.00	15.83	0.00
1.00	106.38	2.08	382.	4657.	4.67	15.83	0.00

P.M.F. OUTPUT DATA
Sheet 9, Appendix C



Sheet 10, Appendix C

APPENDIX D

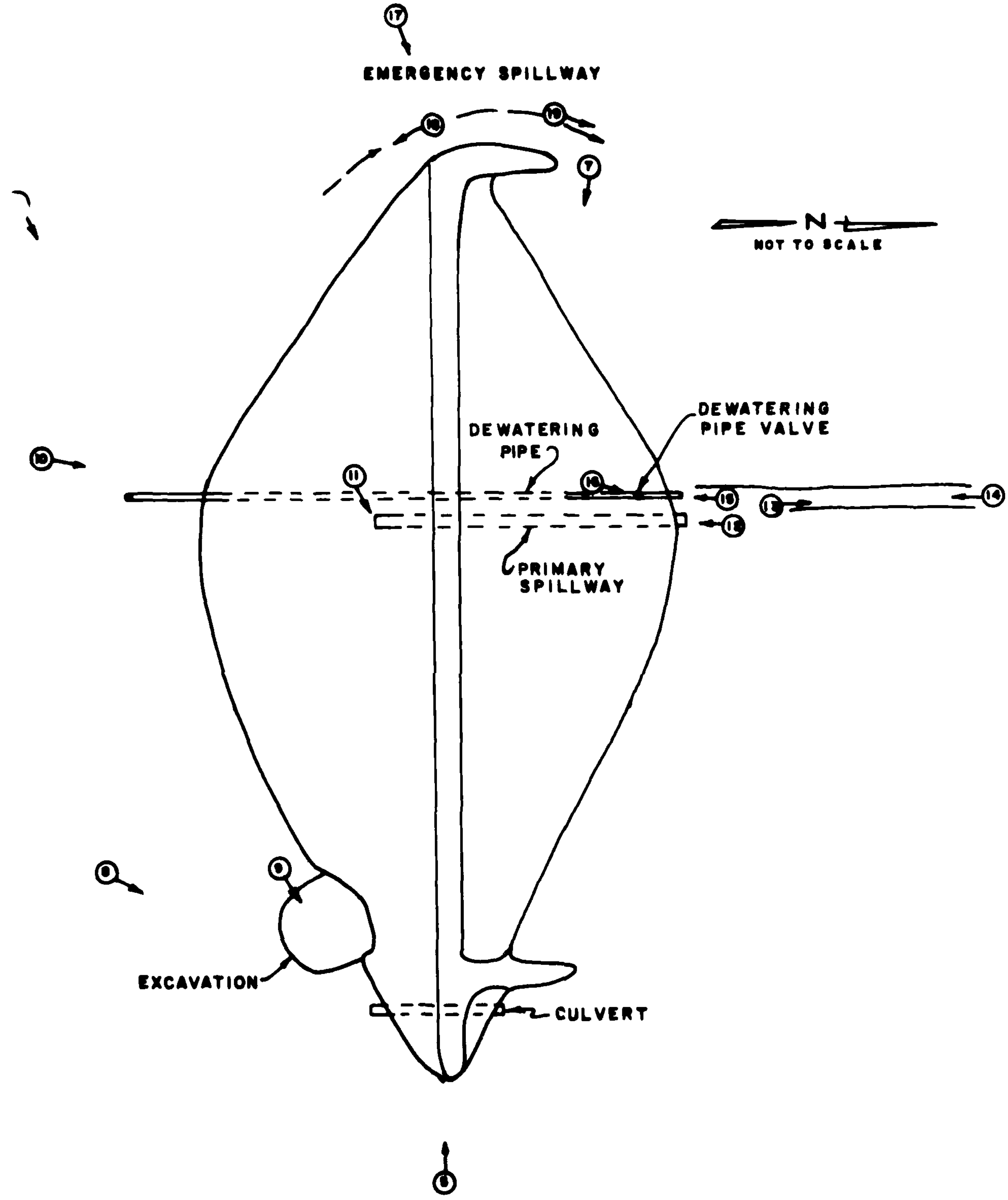


PHOTO INDEX

VON DER AHE DAM

FRANKLIN COUNTY, MO.

Sheet 1 of Appendix D

LIST OF PHOTOGRAPHS

Photo No.	
	Photos Taken 25 June 1979
1.	Aerial - Lake and Watershed Area-Looking South
2.	Aerial - Lake Area and Dam - Looking Southeast
3.	Aerial - Dam - Looking East (Note Excavation West Abutment)
4.	Aerial - Dam - Looking North
5.	Crest of Dam - Looking West
6.	Upstream Face of Dam - Looking Northeast
7.	Downstream Face of Dam - Looking Southeast
8.	East Abutment Excavation - Seepage Area
9.	East Abutment Excavation - Seepage Area
10.	Dewatering Pipe (Foreground); Spillway Pipe (Background)
11.	Inlet - Primary Spillway Pipe
12.	Outlet - Primary Spillway Pipe
13.	Outlet Channel - Looking Downstream
14.	Outlet Channel - Looking Upstream
15.	Outlet - Dewatering Pipe
16.	Downstream Valve - Dewatering Pipe
17.	Emergency Spillway Area - West Abutment
18.	Emergency Spillway - Looking Upstream into Lake Area
19.	Emergency Spillway Outlet - Looking Downstream
20.	Apparent Exit Point of Seepage - Several Hundred Yards Downstream of Dam

Photos Taken 22 May 1980

21. Upstream Face of Dam, Looking East

22. East Abutment Upstream Contact

23. East Abutment Upstream Contact

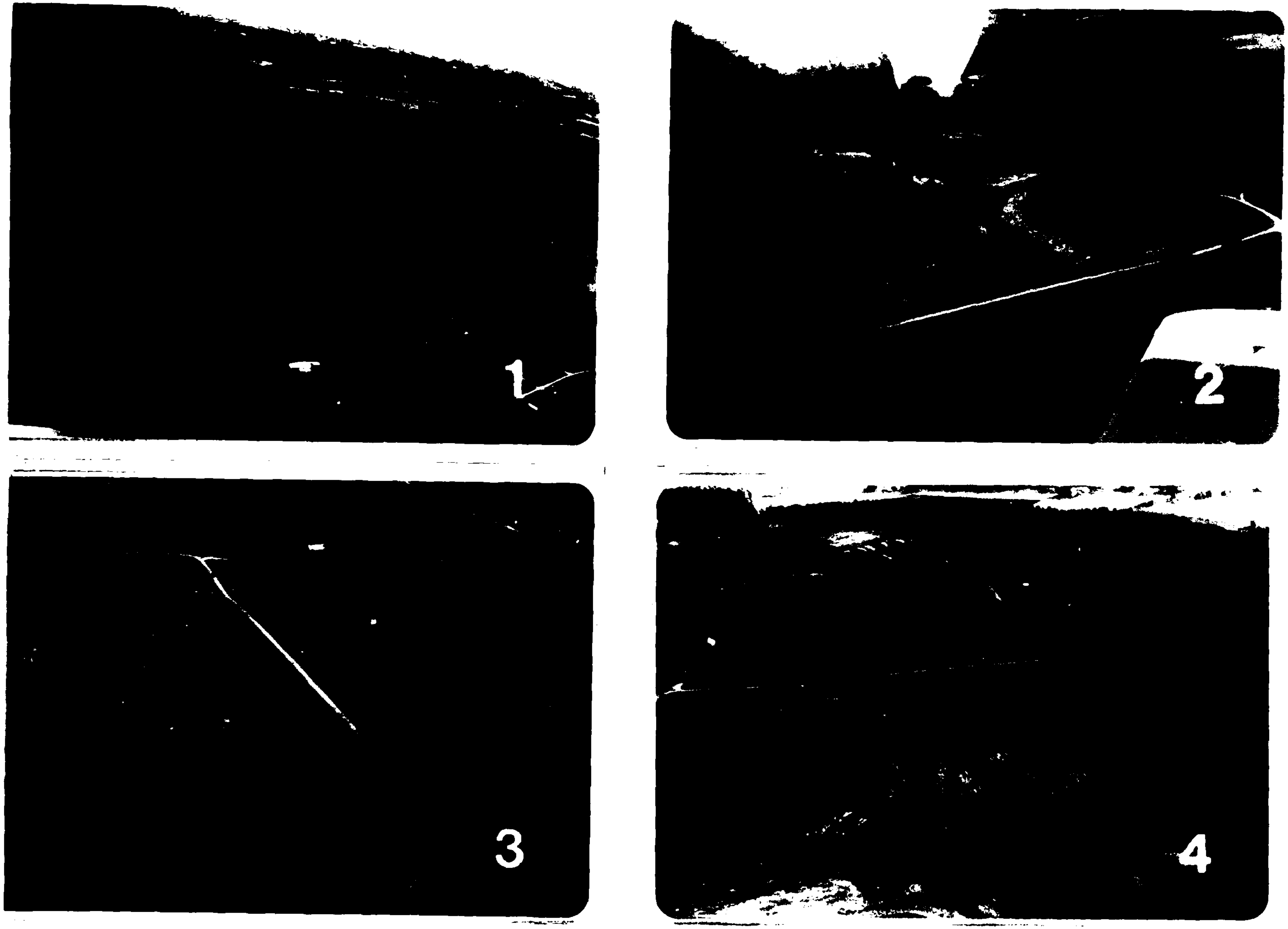
24. Reservoir Area

25. Upstream Face, Looking Southwest

26. Outlet Channel, Looking Downstream

27. Outlet Dewatering Pipe

28. Aerial - Dam - Looking Southwest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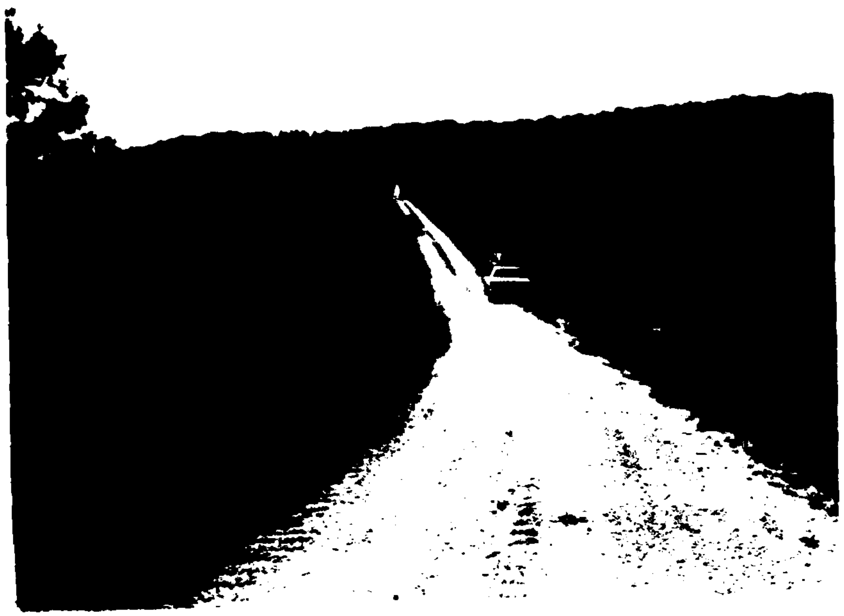

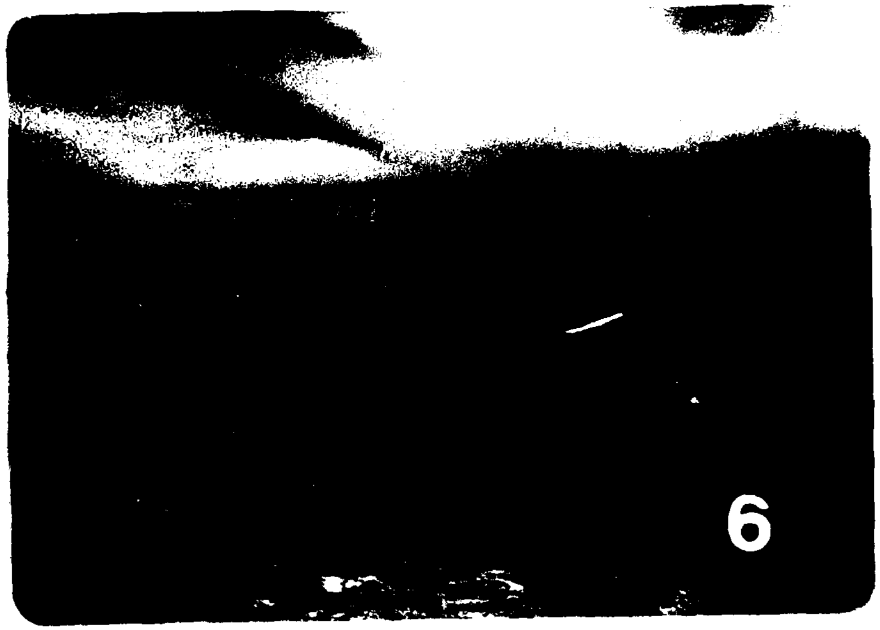
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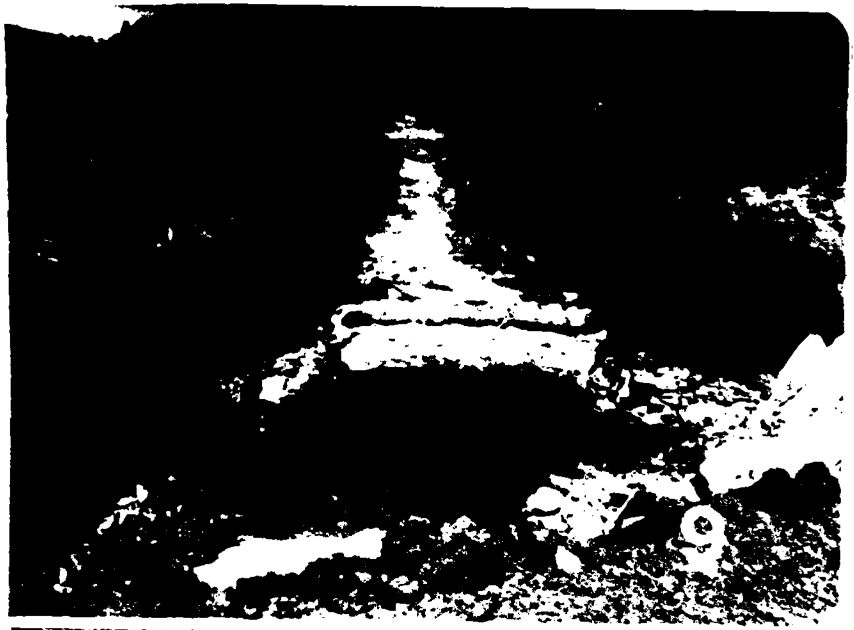
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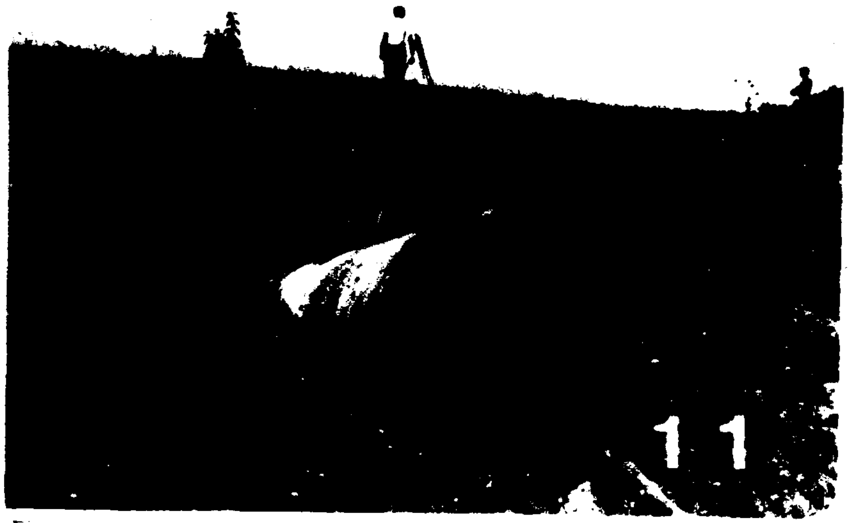
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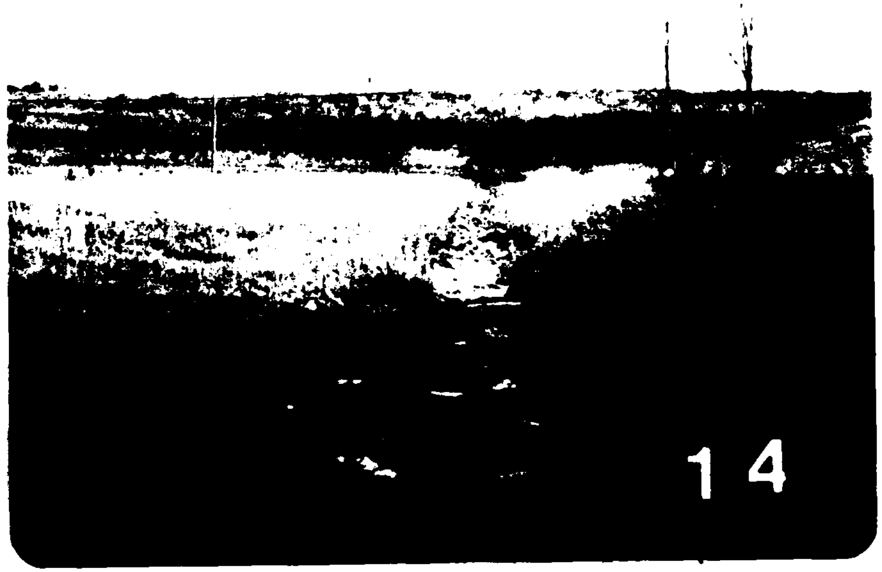
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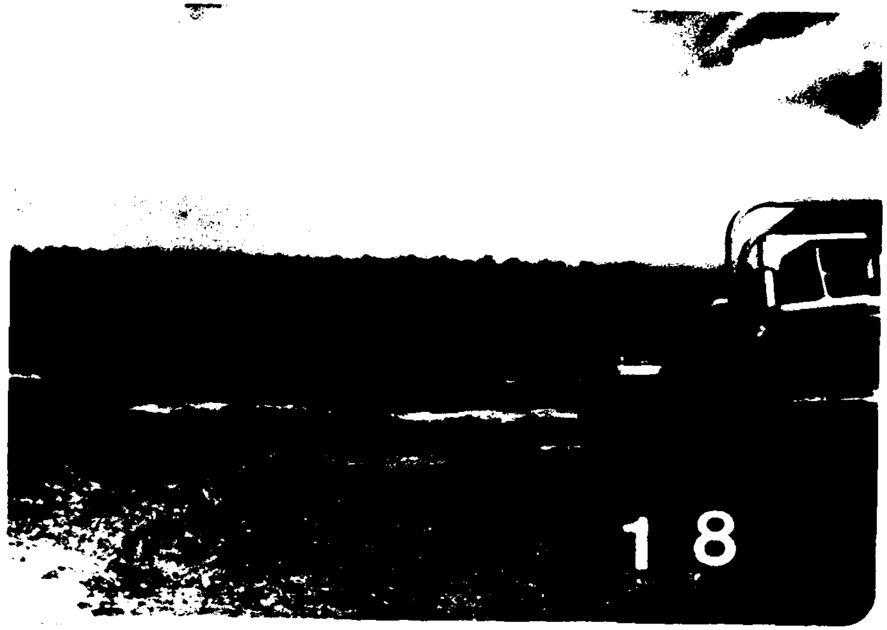
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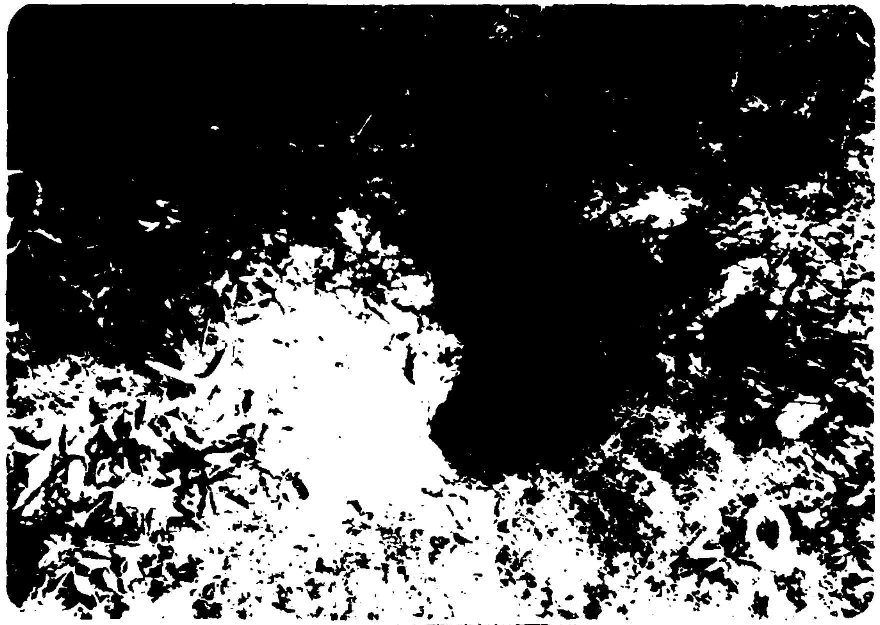
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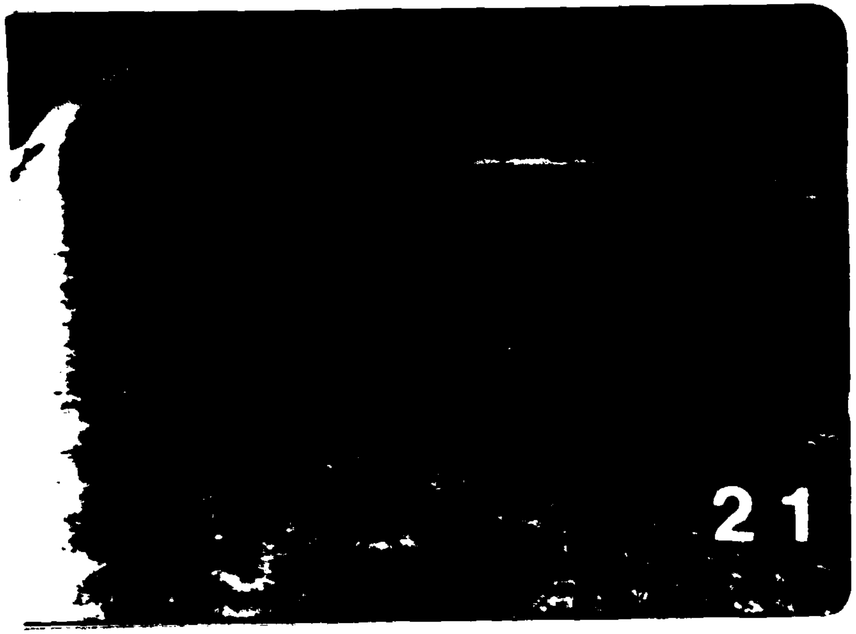
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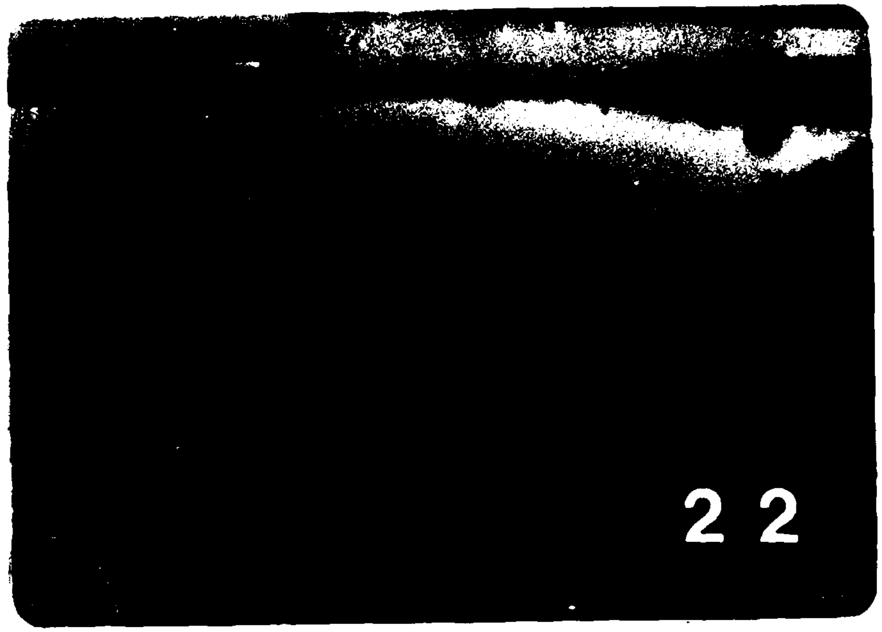
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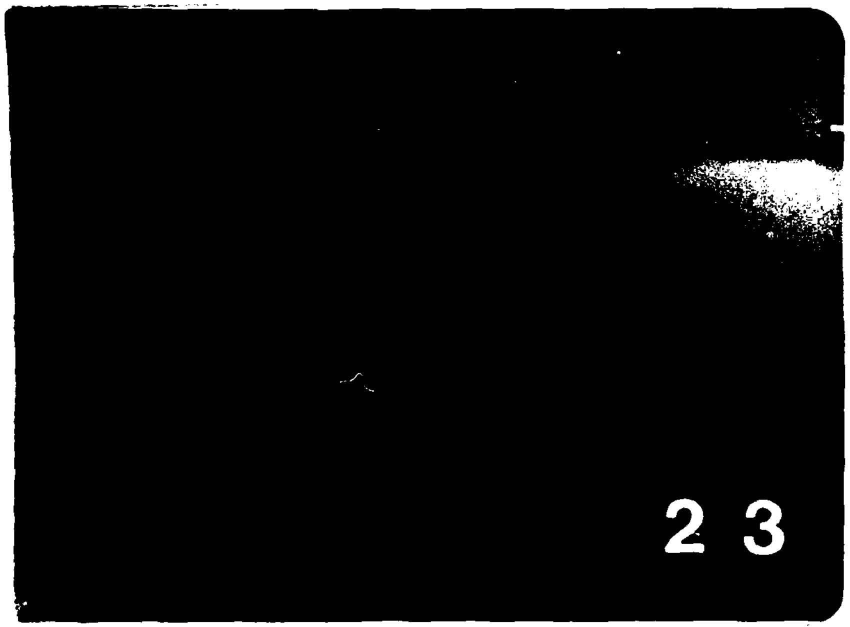
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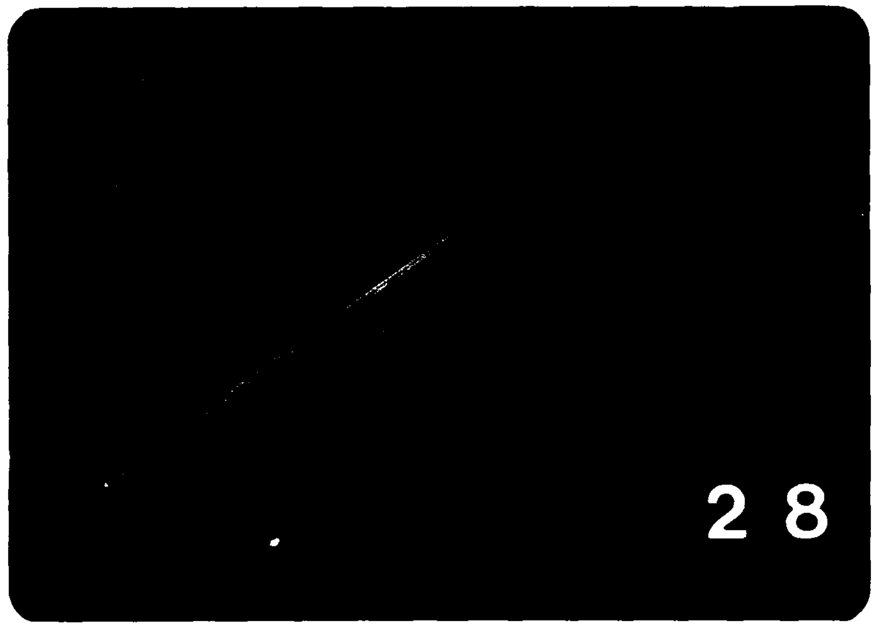
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